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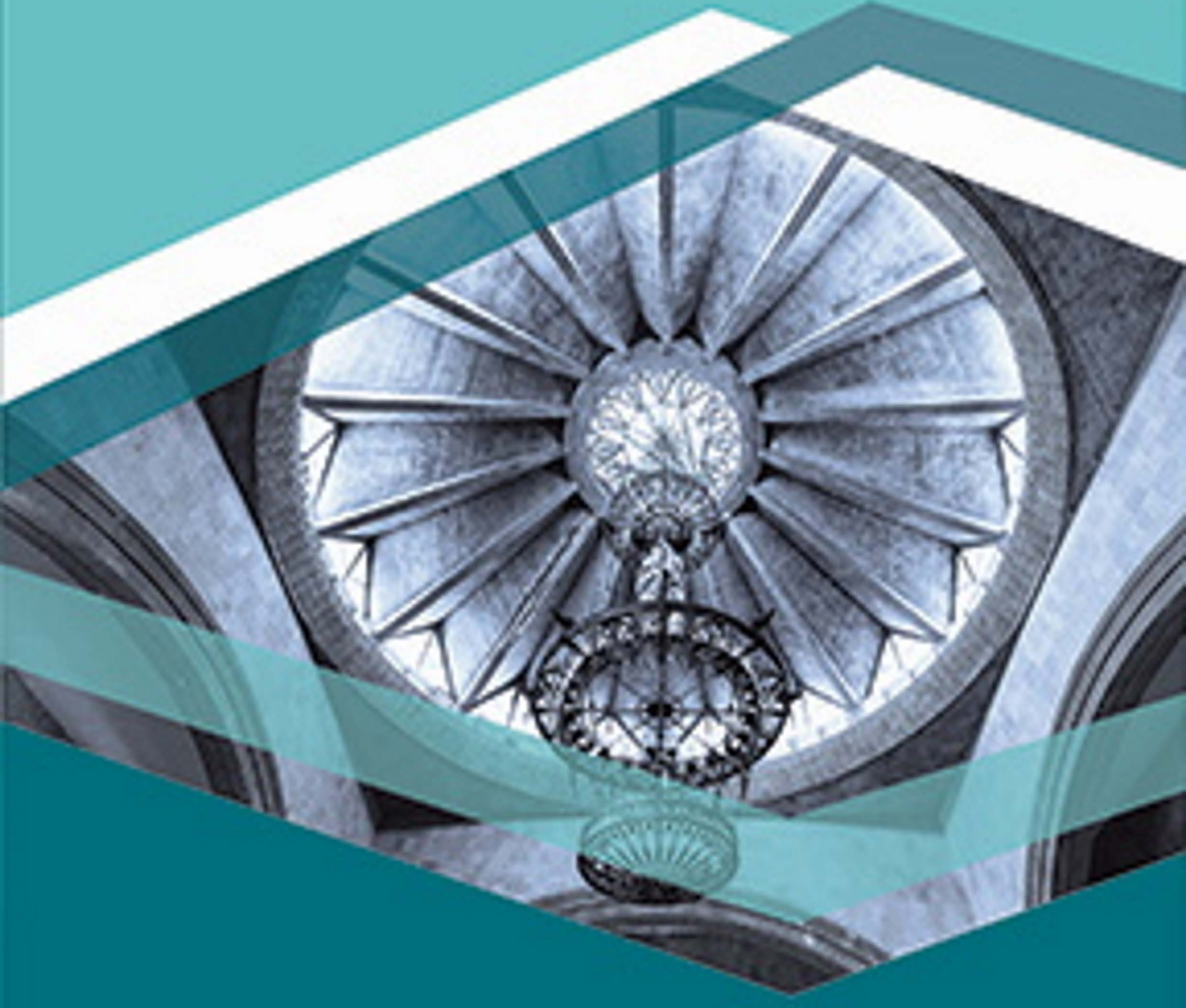

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CONTENT

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CONCEPTUAL APPROACHES TO SOLVING THE ISSUE OF REINFORCING THE ROCK - CUT STRUCTURES OF GEGHARD MONASTERY COMPLEX

The article presents a visual and instrumental research of the technical condition of the main rock-cut structures and their masonry additions, the documentation of their damages (cracks, crevices, destructions and erosions) and deformations, thorough complete laboratory studies of rock samples and their physical and mechanical characteristics, conceptual approaches to preventive and reinforcing measures necessary for the further safe survival of structures, as well as the comprehensive development and implementation of measures to prevent further damages (elimination of causes) and ensure the long-term existence of structures. Based on the analyses carried out, it is recommended to use a ready-made dry mixture mortar of the "Mapegrout" brand produced by the Italian company "Mapei" to fill cracks if necessary. It is available in the market of the country and is successfully used in the reconstruction of tunnels and other underground structures. The issues of compatibility of reinforcing materials with sandstone rock are also considered on the basis of some averaged data of the main decisive physical and mechanical characteristics of the strength and deformation of sandstone.

Key words: *fine concrete, cement adhesive, basalt-fiber, reinforcing, dry mortar "Antique I".*

Introduction

Geghard Monument Complex generally includes rock-cut, as well as attached masonry structures and adjacent rock mass. In addition to the destructive phenomena of decomposition under freezing and weathering of rocks and adjacent rock masses, the complex has also been heavily affected by earthquakes in the region, and the current damage to it is the sum of the aforementioned water factors and seismic effects [1]. At the same time, damages from the former have steadily increased and continue to increase the seismic vulnerability of the Complex. There are many other dangerous damages of various degrees to the rock-cut and masonry additions of the structures. Therefore, for a comprehensive development of the main measures related to the subject, all structures and the adjacent slope cliffs should be subjected to a thorough professional visual and instrumental examination. After a comprehensive analysis of the data obtained, the correct scientific and technical solutions should be proposed.

Materials and Methods

The envisaged works will make it possible to quite subjectively develop and apply a set of technically justified measures for the water protection of the considered monumental structures and the strengthening of the adjacent rock mass, which is of decisive importance for their further safe existence. However, the preliminary studies and observations already conducted allow us to propose the following basic conceptual approaches to the required measures.

1. To protect monumental structures from slope waters of the nearby area, it is necessary to implement slope surface water catchment and safe drainage streams at the bottom of the slope before reaching the structures. For this reason, by involving an environmental specialist in the process and based on the geodetic surveys to be performed on the site, more efficient stream contours should be developed, and in this case minimal interference with local masonry additions and cuttings on the rock should be made, while making

maximum use of the natural slope relief structure. Masonry additions forming the streams should be implemented after removing a weak upper layer of weathered rock in their place, with a mass made of covering of unprocessed rock pieces and fine-grained concrete or cement-sand mortar made with the implementation of calmatron, aquatron additives having good adhesion and ensuring water resistance or another inorganic additive having similar purpose. If the required height of masonry additions is up to *30 cm*, it can be carried out without spring anchors attached to the rock, and if they are much thicker, the addition must be attached to the rock with additional grooving with reinforcement anchors. At the same time, taking into account the known facts (Table) about further easy corrosion of steel reinforcement bars and incomparably higher coefficient of thermal expansion in comparison with the stone, it is recommended to make reinforcement anchors from basalt-fiber composite reinforcement rods of 10, 12, 14 and 16 *mm*, which are already produced in the country and are not so dangerous.

With their lower edge, the reinforcement anchors should be placed in the preliminary perforated and at least 100 *mm* deep holes in the lower harmless part of the rock with an anchoring adhesive, while with their upper edge they should enter to the cement or mortar part of the addition. In case of applying a large number of rock pieces on the rock side of the addition, the upper edge of reinforcement rods can be placed in the perforated holes of such stones too.

The bottom and walls of the natural looking water stream, formed mainly by natural relief lines and small masonry additions and excavations, should then be made water permeable by immersing them in a water-repellent solution of the durable inorganic type GS Pronitrate (or another inorganic liquid substance having a similar function). Water repellent solution should be applied to the specified surfaces at intervals of not more than 10 minutes by spraying 3…4 times or abundantly applying with a brush.

In order to ensure the smooth operation of the catchment and drainage stream for the slope water, it will need current repair services at least twice a year after the spring and fall abundant rainy seasons.

2. On the surface of the slope it is necessary to exclude the possibility of water accumulation in the local catchment pits, causing deep erosion of the rock mass and to provide an automatic rapid flow of surface water to the stream designed to catch and drain water from the slopes. In this case as well, based on the principle of not deteriorating the natural form of the slope, it is necessary to be satisfied with solutions with minimal interference. Depending on the size and depth of the pit location, and with the advice of an environmental specialist, this problem can be solved in two ways or by combining those options.

The first option is to fill a local low land, and *the second* is to create local excavation streams to drain the water accumulating in the low land. In some cases, by the decision of the environmental specialist, a mixed option of partial filling of the low land and of creating a stream for the remaining part may be used. The choice of any of the solutions should be based on the peculiarities of the relief of a particular site and the opinion of the environmentalist, so as not to deteriorate the form of the slope. Moreover, the environmentalist must consider that the existing catchment pits are not at all the original condition of the slope and have arisen as a result of further disproportionate erosion of the rock.

The filling of the local lowlands on the slope should be carried out according to the similar technology as for the addition of drainage streams presented in the previous point removing a weak weathered layer of rock, in the intermediate part, reinforced (using the same non-corrosive basalt plastic reinforcement rod) with the same well adjoining water permeable concrete and lining of local unprocessed rock piece or with a mass formed without it. In the second option, drainage streams of low land sections should be carried out at regular or irregular local excavations on the rock mass. Afterwards, the surfaces of both excavated streams and protected low lands should be treated in the same way: by soaking in the aforementioned water-repellent, long-lasting inorganic liquid.

3. Based on hydrogeological and deep geophysical studies of structures and the adjacent rock mass with the work presented in stage 3 [2], deep rock distortions (layerings, cracks, crevices), as well as from

documenting data obtained by identifying water veins and watercourses and their sources of supply within the stratosphere (location, depth, direction, etc.), rock pieces with different levels of separation from the overall rock mass should be divided into two groups:

1. No longer suitable for strengthening and storage and subject to removal,
2. Suitable for strengthening and preservation.

Safe removal of rock pieces of the 1st group, i.e. priority works on neutralization of the rocky slope adjacent to the structures should be organized with the involvement of specialized services of the Ministry of Emergency Situations of the Republic of Armenia. Moreover, healthy types of rock pieces up to 30 *cm* in size can be accumulated for the implementation of slope streams and other masonry additions.

To reinforce the rock pieces of the 2nd group, first of all it is necessary to carry out a set of measures to blind the sources of upper layer waters in these areas, as well as to restore the density and stability of the rock mass fragmented from the existing deformations.

It should be noted that particular care should be taken with the materials to be used in the restoration of such valuable monumental structures. Given the currently widespread organic polymeric regeneration substances, the relatively short duration of their effective action over time due to the known tendency to depolymerization and aging, their use should be limited in valuable monumental structures subject to long-term storage, and in extreme cases their use should be limited to only indoor environments not exposed to factors that promote rapid aging (sunlight, significant temperature fluctuations, etc.). Based on the same principle, corrosive black metals should be avoided in such structures or should only be used in areas where there are absolutely no corrosive conditions. In this regard, we believe that stainless steel and other metals or stainless non-metallic reinforcing materials should be used in monumental structures to provide high durability, such as fiberglass and basalt-fiber composite reinforcement, carbon fiber ropes, bands and canvas, etc. Some averaged data on the physical-mechanical decisive key characteristics of strength and deformity of these materials and the observed sandstone rocks are shown in the Table.

Now, based on the data in the Table, the comparability of the listed possible reinforcing materials with sandstone rocks under interference has been considered. It is obvious that when rigid steel reinforcing bars with a high deformation modulus is inserted into the rock, especially in case of dynamic influences, it can become a source of pressure concentrations in the rock with rather low deformation modulus. On the other hand (Table), both black and stainless steels have some incompatibility with the existing rock material in terms of thermal deformation. In this regard, it is obvious that the nonmetallic reinforcing bars listed in the Table, in addition to the positive property of non-corrosion, has a relatively low modulus of deformation and a certain flexibility, as well as close characteristics of thermal deformity have favorable combination characteristics with the rocks and the ability to easily adapt to the environment.

To restore the density and stability of the rock mass fragmented from the existing deformations, it is advisable to implement the following solutions:

3a) Attach large rock pieces of the 2nd group surface layers, which are separated from the general rock mass, but are to be preserved, to the lower dense rock mass creating groove anchors. To do this, a hole with a diameter of 50 ... 150 *mm* and at least 100 ... 300 *mm* straight deep, depending on the mass of the attached piece, should be drilled on the given rock piece and into the lower rock mass with a special drilling instrument, and a reinforcement cement groove with basalt fiber reinforcement frame should be implemented in them. Grooves should be concreted with the aforementioned adhesive and water permeable non-shrinking heavy concrete mix not less than B20 with the adequate implementation of corresponding expanding additive. The number of grooves on a given rock piece depends on the planned size and weight of it, which should be calculated. The adjacent separated rock pieces should then be connected at the top by means of a carbon fiber rope or band connecting the tips of their grooves. The latter must be placed in a pit drilled on the surface of the rock and covered with the above mentioned type of concrete and unprocessed rock covering.

3b) At the most accessible level the lower marks of cracks and crevices with contours of rock pieces separated from the overall rock mass and of all cracks and crevices on the rock mass as a whole, using a syringe first inject a water-resistant adhesive containing a liquid consistency or fine-powder grained additives, which easily penetrate into the crumbling interlayers, consolidate the flowing mass in it and glue the separated parts of the rock mass together, for example, from the above-mentioned type GS Pronitrate, Aquatron or Calmatron, Cerezite and other materials of similar function. If necessary, ready-made dry mixture mortar "Mapegrout" produced by the Italian company "Mapei" available in the market of the country can also be used to fill the cracks.

3c) After the initial measures listed above, all cracks and crevices in the rock should be firmly filled. Depending on the size of the openings in the latter, their filling should be carried out from the injection of fine adhesives to the application of gravel up to 20 *mm* grain size and with the insertion of gravel plasticized self-flowing fillable concrete mix with plasticizer [3-5]. Both adhesives and concrete must be non-shrinking, water resistant and well adhered to the rock mass through the use of appropriate additives. The upper surfaces of large cracks should also be covered with local unprocessed rock pieces. It should be noted that the covering with the same rock type of both the given cracks and the additions of previous points to be implemented, besides providing the same texture of the slope, firstly has a well-founded technical goal of proportional weathering of the rock surface and not creating local pits again in the future.

3d) Interlayer waters blinding works of deformed rock masses adjacent to main monumental structures should be carried out by appropriate measures developed by hydro-geological and geophysical studies based on the identified data on their locations and features.

Table. Relative average indices of firmness and deformity of sandstone rock and some reinforcing materials

Name of the materials		Compressive (tensile) strength, *MPa*	Average density, *kg/m³*	Modulus of elasticity, E, *MPa*	Thermal expansion coefficient, $10^{-6} \cdot {}^{o}C^{-1}$
Steel reinforcing bars	Class A-I	225 (225)	7850	$2.1 \cdot 10^5$	13…15
	Class A-II	280 (280)	7850	$2.1 \cdot 10^5$	13…15
	Class A-III	355…365 (355…365)	785	$2.0 \cdot 10^5$	13…15
	Class A500C	500 (435…455)	7850	$(1.9…2.0) \cdot 10^5$	13…15
	B_p-I class	360…375 (360…375)	7850	$1.7 \cdot 10^5$	13…15
Stainless steel	Ferritic	190…310 (190…310)	7850	$(1.94…2,0) \cdot 10^5$	16.6
	Austenitic	190…310 (190…310)	7850	$(1.93…2,0) \cdot 10^5$	17.3
Non - metallic reinforcing bar	Basalt Plastic	- (700…1300)	1200	$0.6 \cdot 10^5$	9…12
	Glass Plastic	-(600…1200)	1900	$0.45 \cdot 10^5$	9…12
Carbon fiber band	Sika carbondur-S	-(3050)	-	$1.65 \cdot 10^5$	-
	Sika carbondur-M	-(2900)	-	$2.1 \cdot 10^5$	-
	Sika carbondur-H	-(1450)	-	$3.0 \cdot 10^5$	-
Sandstone		15…30	1350…1450	$0.14 \cdot 10^5$	7…11

4. Given the variety of mostly water-related damages in structures, the problems of eliminating each of these and strengthening the structures as a whole should be approached individually, according to the decisions made as a result of a thorough professional study.

As for the problem of restoring and putting back to service bearing walls and coverings of rock-cut structures that were originally dense but are now fragmented and cracked and do not fully perform their role, the density of the deformed stones must be restored after removing the weak surface layers of cracks and crevices and after injecting water resistant and non-shrinkable during fastening mortar and mastic materials giving good adhesion to the rock into the healthy rock inside them, which has adequate characteristics with modulus of deformation and coefficient of thermal expansion.

In this case, only special cement-based fine-grained mortars close to the existing rock material can be used as injection materials. Cement mortars can be produced with the complex implementation of high quality quartz sand, Portland cement and non-shrinkable, adhesive calmatron, aquatron or other additives with similar function, as well as plasticizers, adhesives and other additives having the same purpose [6,7]. Grain composition of the mortar should be adjusted according to the size of the crack or gap to be filled. In addition, in the case of microcracks adhesive mastic, made on the basis of fine grained sand and the same additives should be applied.

In rock-cut structures, walls and enclosures of ground anchoring parts damaged by prolonged wetting and deconstruction under freezing require a special approach. In this case, before dealing with the damaged elements themselves, you must first eliminate the root cause of the damage - abundant water infiltrating the bottom of the structure. Speaking about the rock-cut church, for example, it should be noted that besides the aforementioned blinding of random interlayer water streams above the structure, here measures should be taken to prevent the wetting of the dense rock mass of the structure's ground layer as well as the constant wetting of ground anchoring parts of its walls with capillary action from the open basin of the stream operating in the internal area. To do this, it is necessary to temporarily collect spring water directly from the outlets and take it out through pipes, drain water from the existing pool and watercourses using specially installed pumps. Then the surfaces of the latter related to water should either be blinded by absorbing reliable and durable water-repellent mortars, or covered with water permeable reinforced concrete pits reinforced with non-corrosive basalt-fiber reinforcing bars and with the implementation of streams, which is preferable. Reinforced concrete pits and streams can be covered with pieces of unprocessed rocks from the same rock along with covering with concrete. Regarding water outflows from an existing stream, it is possible to create these waters in the back of the walls by local drilling in those parts of the property, assembling a controlled permeable initial basin from which the water flows inside the church through a permeable pipe.

After carrying out the aforementioned water protection measures, it is necessary to give time for the sufficient drying of the stone mass of the church floor, after which only to begin restoration and reinforcement work of the lower parts of the walls and pillars deconstructed under freezing. The restoration of the lower loaded part of this responsible constructive part of the structure with the previous section should be done with an additive capable of working with a healthy stone cutting. Therefore, the most important prerequisite for additive material is to provide strength and thermal deformation and strength values similar to the existing rock material, which in the case of the given rock type, as well as in the case of cracking and filling, can be performed with a quality quartz sand, Portland cement and calmatron, aquatron or other similar function, which provides good adhesion and is non-shrinking, as well as with special mixtures made with the complex use of other target additives, simultaneously providing high adhesion (for example, the highly effective proprietary adhesive addition "Planicrete" by "Mapei"). Before carrying out restoration work, carefully remove the weak surface layer deconstructed under freezing from part of the structure and thin reinforcement, consisting of basalt-fiber thin anchors 2...3 *mm* thick attached to it and basalt fiber reinforcement net, put it on the hard rock freed from it, as well as in the drilled holes in the floor. After that, clean this part of the dust, rinse it with a jet

of water and use pressure plastering to install a special fine-grained concrete with the above-mentioned characteristics.

If desired, to ensure greater reliability and durability of the structure, after sufficient reinforcement of the reinforced concrete layer, a high-strength carbon fiber band (Table) can be applied with epoxy adhesive and then covered with protective stone. Restoration to the original dimensions of the remaining sections of surface layers of the roof, walls and pillars deconstructed under freezing should be carried out in the same way as described above, using the same materials. As for the issues of reconstructing rock masonry constructions implemented with lime-sand mortar and now having damages added to the rock-cut parts of the structure, then, in their restoration work, both for upper masonry layer and for injection, it is necessary to use the assortments of complex dry lime-sand mortars prepared by the Italian company "Mapei" for the purpose of special restoration works using special complex additives or on the basis of local raw materials to develop complex lime-sand compositions providing the proposed technical and economic characteristics of the mentioned dry mixtures. Such is "Antique I" dry mortar, made with the mixture of hydraulic lime, eco-pozzolana, natural fine and ultrafine sand, a range of special targeted additives. It is intended for injection into cracks, crevices and voids of damaged walls made of lime-mortar to restore their density and structural integrity.

Conclusion

Complex work on the elimination of the damages of water impacts, as well as the restoration and reinforcement of the original monumental structures with rock-cut and masonry additions [8-10] should be carried out by a complex implementation of the data recorded by thorough professional technical studies and their adequately developed measures. Moreover, both the choice of materials to be used and the solutions, must be guided by the above-mentioned provisions, which do not disturb the creation of the structure and ensure great durability and reliability, and in general by the preservation of internationally accepted standards for the restoration of monumental structures.

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ANALYSIS OF STRUCTURE DISPLACEMENT FORMED AS A RESULT OF TORSION IN CASE OF USING RUBBER-METAL SEISMIC ISOLATORS

The study of a building with a complex planning solution, rubber-metal laminated seismic isolation supports (hereinafter referred to a RMLSIS) placed at the foundation level and with a reinforced concrete frame-braced system is presented, taking into account the displacement of the structure formed as a result of torsion. The analyses were conducted with the finite element method. The calculation schemes were modeled using the "Lira-SAPR" software. The displacement of the structure formed as a result of torsion of buildings having the same planning solution, different number of floors as well as with and without RMLSIS is estimated. The results obtained show that the displacement values of the structure formed as a result of torsion increased about 40% in buildings without RMLSIS and 25% in buildings with RMLSIS.

Keywords: *rubber-metal support, reinforced concrete structure, seismic isolation, finite element method, torsion.*

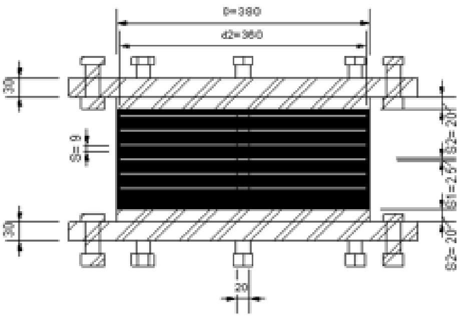
Introduction

In recent years, seismic isolation supports have become widespread. In the case of low-rise buildings, requirements of seismic codes can be fully met, but in the case of multistory buildings, the number of problems increases. In particular, the displacement of the structure formed as a result of torsion plays a significant role in the further operation of the building. Taking into account the fact that the Republic of Armenia is in the seismic zone [1], it is necessary to ensure the reliability and durability of the bearing systems of the buildings according to earthquake resistance, at the same time, taking into account the impact of the torsion on the building.

Surveys of buildings after the earthquake from the mid-twentieth century revealed damages caused by the torsion discovered by many local and foreign researchers. It should be noted that after the earthquake of 1985 in Mexico City, many buildings, destroyed because of torsion, were discovered. The damages caused by torsion accounted for 42% of the total number of destroyed buildings [2].

Materials and Methods

By now, indicators of the economic efficiency of seismic isolation supports have been studied and it has been shown that the use of seismic isolators is not very profitable [3-5]. The use of RMLSIS results in increased values of displacements of structures formed as a result of torsion. In particular, the displacement of the structure is significantly increased compared to the case without the use of RMLSIS [6,7]. The characteristics and geometric dimensions of RMLSIS used in RA are given in Table 1 and Fig. 1[1].



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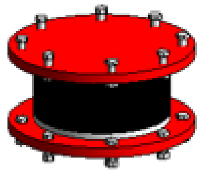


b

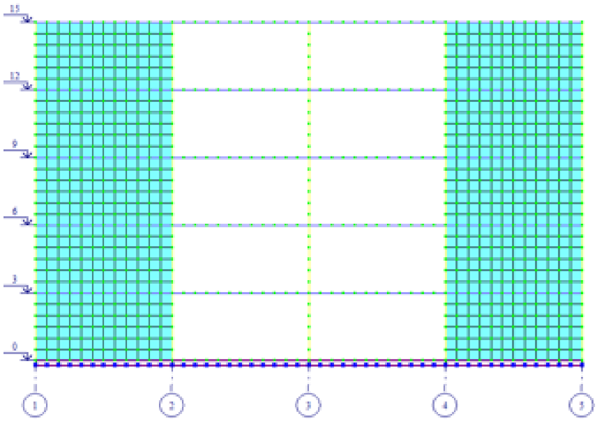
Fig. 1. *RMLSIS used in RA, mm. a - geometric appearance, b - three-dimensional appearance*

[1] HST 261-2007 Seysmamekusatsman shertavor retinametaghakan henaran. Tekhnikakan paymanner, Yerevan, 2007, p.17 (in Armenian).

Table 1. Characteristics of rubber-metal laminated seismic isolation supports

Support diameter (D), *mm*	380.0
Flange diameter (d), *mm*	560.0
Number of rubber layers, (NR)	14
Number of metallic layers, (NS)	13
Thickness of metallic layers (tS), *mm*	2.0
Thickness of rubber layers (tR), *mm*	9.0
Height (H), *mm*	206.0
Horizontal stiffness (KH), kN / *mm*	0.81
Maximum displacement in the horizontal direction (L), *mm*	280.0
Vertical stiffness (KV), kN / *mm*	300.0
Maximum allowable vertical load (P), *kN*	1500.0

Five, seven and nine - story buildings of the same planning were considered (Fig. 2).



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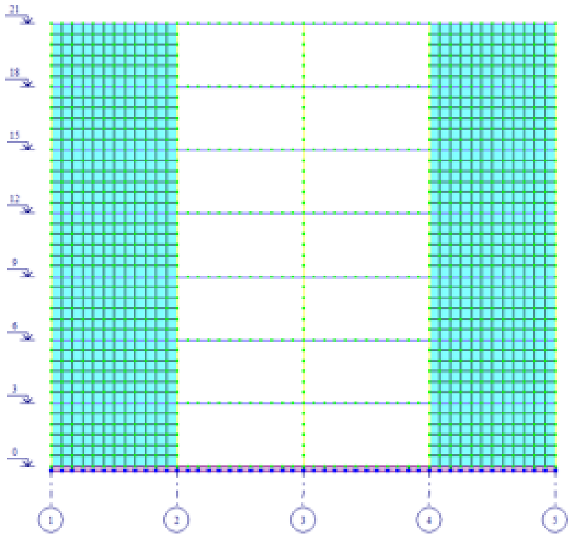

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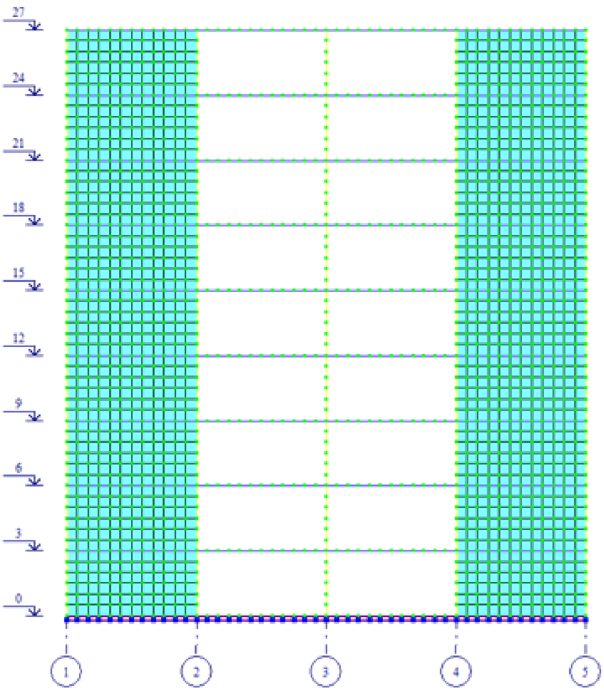

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Fig. 2. *Five-story (a), seven-story (b) and nine-story (c) building sections with "D" axis*

Bearing elements in the structure are made of monolithic reinforced concrete, sections of which are presented in Table 2.

Table 2. Baseline data required for analysis

Geometric dimensions of load-bearing elements (*cm*)	
Monolithic columns	50x50
Monolithic beam	40x60
Monolithic r/c walls	20
Roofing tile	20
Heavy concrete	
Concrete type/class	B25
Average density, kg/m^3	2500
Elasticity modulus, (Eb), *MPa*	30000
Compressive calculation strength (Rb), *MPa*	14.5
Reinforcement bar	
Reinforcement bar type/class	A-III (A400c)
Tensile calculation strength (Rs), *MPa*	375
Reinforcement bar elasticity modulus (Eb), *MPa*	200000
Compressive calculation strength (Rsc), *MPa*	375

The calculation is carried out taking into account the own weight of the structures, permanent, temporary, long-term and short-term loads and seismic impacts (in the X and Y directions) [8-10].

All loads were accepted according to construction codes. constant: q_1 = 7519 kN/m^2, temporary long – q_2 = 0.36 kN/m^2, temporary short – q_3 = 1.44 kN/m^2, on the roof (constant: q_4 = 6.3 kN/m^2, snow load: q_5 = 0.98 kN/m^2). Calculation of structural elements was carried out on the main and special combinations of loads taking into account horizontal seismic loads. During determination of seismic loads, combination coefficients were accepted: constant - 0.9, temporary long-term - 0.8, temporary short-term - 0.5. Main coefficients of building calculation are accepted according to RABC II-6.02-2006 (RA Building Codes) normative document[2].

Results and Discussion

The calculations were carried out by the finite element method using the "Lira-SAPR" software[3] [11-15]. Six problems were considered in the calculation, in three of which RMLSIS are installed at the level of the foundations of five-, seven- and nine-story buildings (Fig. 3), and in three other cases the buildings were without RMLSIS.

The number of RMLSIS has been determined according to the maximum allowable horizontal displacement (L) and vertical loading (P) conditions, as well as the technical and economic conditions have been taken into account. In Fig. 4 the displacement calculation scheme of the building formed as a result of torsion is presented.

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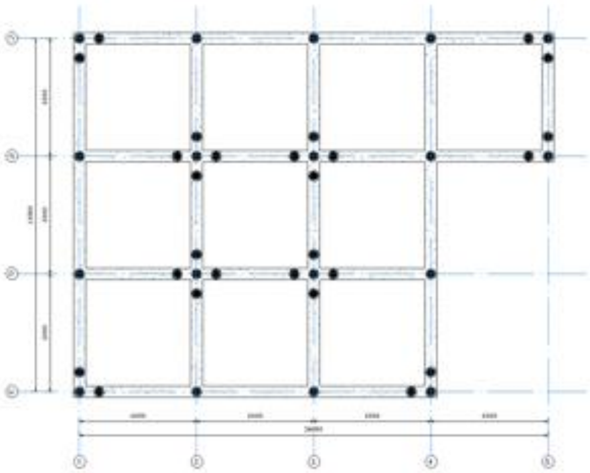
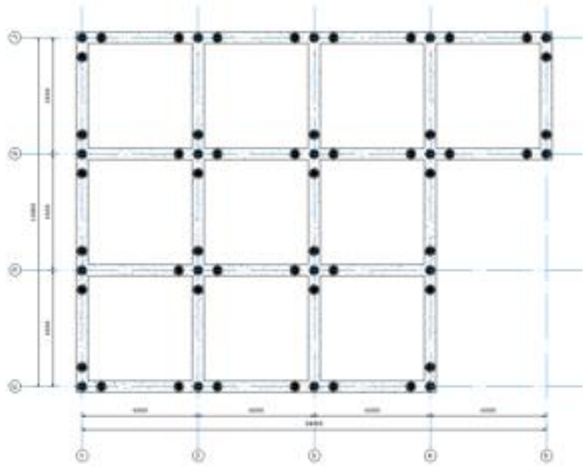
[2] HHSHN II-6.02-2006 Seysmakayun shinararutyun. Nakhagtsman normer, Yerevan, 2006, p.67 (in Armenian).

[3] SNiP 2.01.07-85*. Nagruzki i vozdejstviya, FGUP CPP, Moscow, 2005, p.44 (in Russian).
SNiP 2.03.01-84*. Betonnye i zhelezobetonnye konstrukcii, Moscow, 1998, p.80 (in Russian).

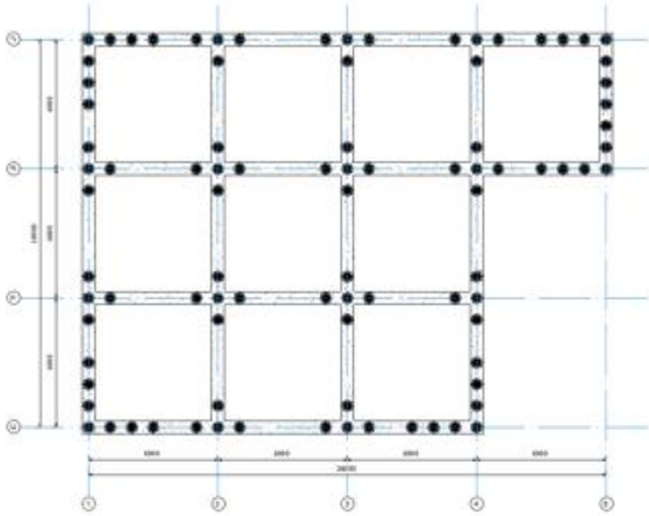
The comparative graphs of the calculation results are summarized in Fig. 5 and Fig. 6.



a



b



c

Fig. 3. *RMLSIS layout planning of five-story (a), seven-story (b) and nine-story (c) buildings*

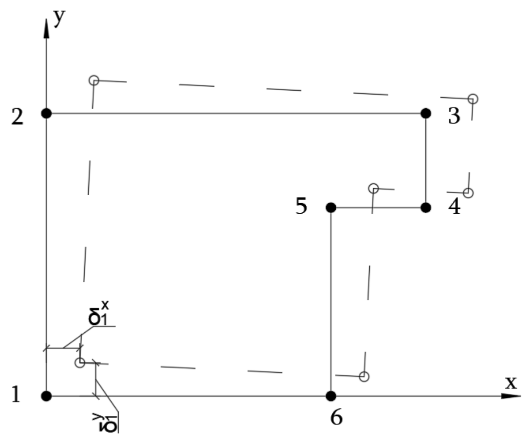
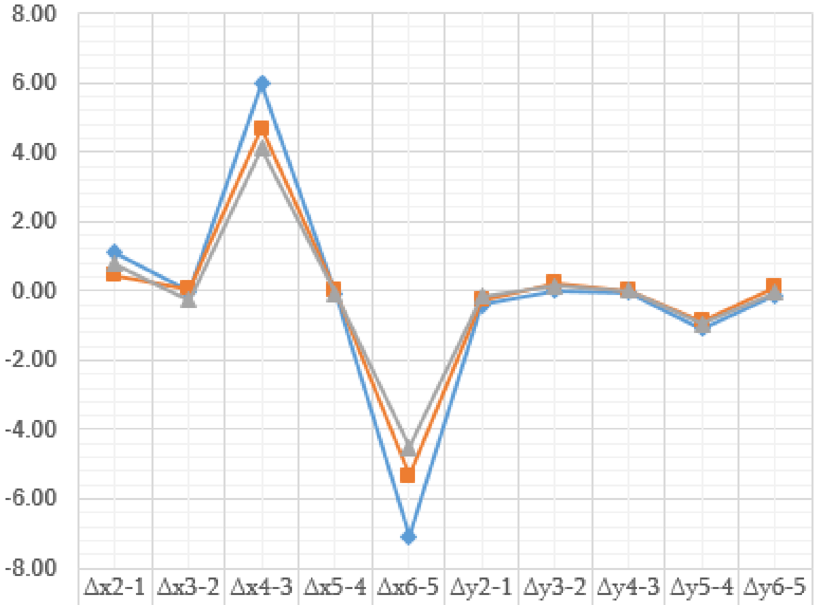


Fig. 4. *Calculation scheme of displacement of the building formed as a result of torsion,* δ_1^x, δ_1^y *– displacement in x and y directions in point 1*

The displacement of a structure formed as a result of torsion can be calculated for each section using the following formulas:

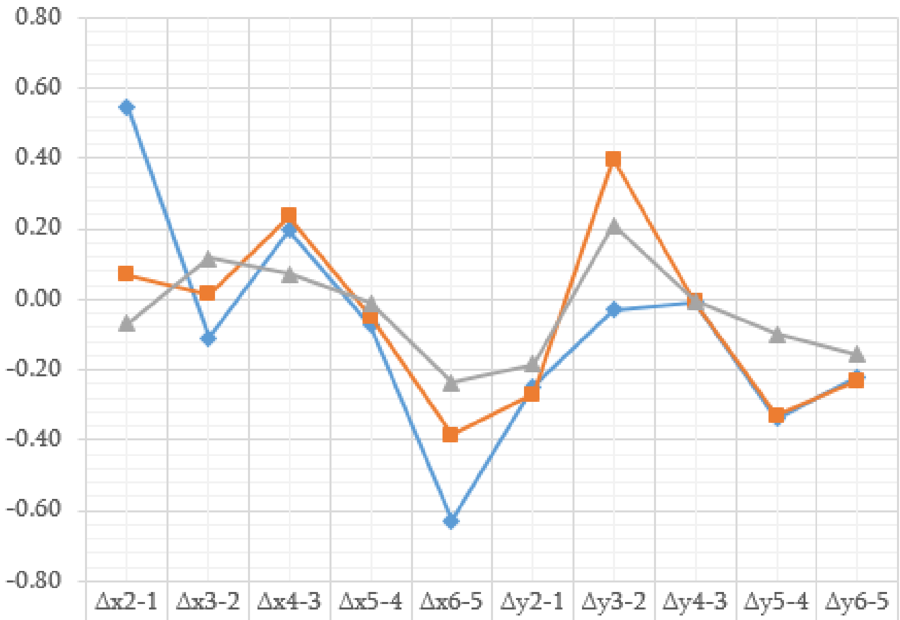
$$\Delta x_{2\text{-}1}=\delta_2^x - \delta_1^x, \tag{1}$$

$$\Delta y_{2\text{-}1}=\delta_2^y - \delta_1^y. \tag{2}$$



	Δx2-1	Δx3-2	Δx4-3	Δx5-4	Δx6-5	Δy2-1	Δy3-2	Δy4-3	Δy5-4	Δy6-5
In a nine-storey building with RMLSIS	1.09	-0.02	5.97	-0.07	-7.09	-0.41	0.00	-0.04	-1.08	-0.14
In a seven-storey building with RMLSIS	0.45	0.06	4.67	0.01	-5.32	-0.28	0.22	0.01	-0.87	0.11
In a five-storey building with RMLSIS	0.77	-0.27	4.12	-0.07	-4.50	-0.16	0.13	0.00	-0.95	-0.04

Fig. 5. *Comparative graph of results of displacement values of buildings with RMLSIS formed as a result of torsion*



	Δx2-1	Δx3-2	Δx4-3	Δx5-4	Δx6-5	Δy2-1	Δy3-2	Δy4-3	Δy5-4	Δy6-5
In a nine-storey building without RMLSIS	0.55	-0.11	0.20	-0.07	-0.63	-0.25	-0.03	-0.01	-0.34	-0.22
In a seven-storey building without RMLSIS	0.07	0.01	0.24	-0.05	-0.39	-0.27	0.40	-0.01	-0.33	-0.23
In a five-storey building without RMLSIS	-0.07	0.12	0.07	-0.01	-0.24	-0.18	0.21	0.00	-0.10	-0.16

Fig. 6. *Comparative graph of results of displacement values of buildings without RMLSIS formed as a result of torsion*

Conclusion

From the calculation results, it becomes clear that as a result of the use of the RMLSIS, the displacement values of structures formed as a result of torsion are significantly decreased. The consideration of this problem is very important to accurately observe the further problems of the operation of buildings and structures. The result of the analysis shows that the displacement values of the structure formed as a result of torsion increased about 40% in buildings without RMLSIS and 25% in buildings with RMLSIS.

Summing up the results obtained, it can be noted that this problem should be taken into account when designing buildings and structures.

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THE IMPACT OF THE CONSTRUCTIVE SOLUTION ON THE FORMATION OF THE PORTAL IN THE ARCHITECTURE OF ARMENIAN CHURCHES OF 4TH-7TH CENTURIES

The article refers to the issues of decorative elements in architecture. The aim of the paper was to reveal the interaction peculiarities of the constructive and artistic solutions of the formation of the portals of the Armenian churches of 4th-7th centuries. The analysis of the process of formation allowed for carrying out the typological classification of the portals from the point of view of the transformation and development of the lintel construction. The classification showed that the portal created as a decoration of the entrance, as a result of the evolution of compositional-structural components was transformed into a unified constructive-artistic element, which has become the main type used in the later periods of the Armenian medieval architecture. The revealed features of the interaction of decorative and structural elements can be useful in the field of heritage studies and in the development of further works on the formation of decorative elements.

Keywords: *portal, structural, decorative, element, architecture, development, formation, churches, Armenian, 4th-7th centuries.*

Introduction

The development of medieval Armenian architecture according to the periodization accepted in the scientific circles is classified into three periods: 4th-7th - Early Middle Ages; 9th-11th and 12th-14th centuries - Developed Middle Ages and 17th-19th centuries - Late Middle Ages [14]). The latter are conditioned by political, public and socio-economic events, which were important for the history of the country and have left a certain mark on the development of the architecture and the art.

The period of 4th-7th centuries takes a special place in this context. This period in the history of Armenia is characterized by significant changes in the life of the country. In the 3th-4th centuries, feudalism replaced the slave-owning system. Feudal-princely dynasties were strengthened, inheritance rights were defined. In parallel with the new socio-economic and political conditions, the pagan faith gave way to Christianity, which in 301 was recognized as the state religion of Armenia. Under these circumstances, in parallel with the civil structures the construction of worship buildings, which were designed to serve the requirements of the new religion, was in full swing. And it was during the 4th-7th centuries that the main types of churches were developed and brought to life, which are the basis of the entire medieval religious architecture of Armenia [35, 38, 13, 37, 26].

As it has been noted by Sh. Azatyan, the architecture of Armenian churches, among other qualities, stands out for its structural meaningfulness and logical moderation of decoration [3]. At the same time, the fact that the portal is one of the important symbolic components of medieval church structures is undeniable [9, 2, 5, 25]. In the general context of the mentioned, it can be noticed that the portal, which became the main element of decoration in the architecture of the churches of medieval Armenia, is of interest both in terms of structure and aesthetics. And although the constructive-artistic solutions throughout the entire Middle Ages generally stand out with the logic of indissoluble unity, that integrity passed its unique way of formation in the early medieval period. This paper is dedicated to the coverage of the interrelationships between structural and artistic solutions in this complex process.

The aim of the article is to analyze the process of formation and evolution of the architecture of the portal of the Armenian churches of 4th-7th centuries and to reveal the interaction peculiarities of the constructive and artistic solutions within the framework of its formation.

The work was carried out on the basis of field observations, as well as research of published and archival materials on the topic, by scientific methods of theoretical research, comparative analysis, classification and generalization.

The elaboration of the article was based on the separation of certain problems related to the topic and the study of scientific literature on them. The following issues have been examined: the essence, peculiarities and periodization of the Armenian medieval architecture [35, 40, 37, 26, 14]; the architecture of specific Armenian temples, historical information on them, measuring materials and restorations [31, 36, 21, 27, 17, 8, 34, 24, 16]; the architecture, decoration and artistic elaborations of the Armenian medieval portals, the restorations developed on the basis of the comparative analysis and the excavation results [13, 31, 41, 36, 27, 23, 33, 19]; the results of the field observations of the Armenian portals of the 4th-14th centuries, general analysis, systematization, chronological and typological classification [3]; graphic and analytical information on the monuments outside the Republic of Armenia [38, 37, 19, 10]; general issues on portals in the medieval architecture [9, 2, 1, 5, 25] and in particular, the structural and artistic issues of the lintel [28, 6, 4]; some features of the formation of the structural solutions for coverings [12, 20, 22]; structural issues and stone architecture [7, 18, 30, 32, 29, 11]; implementation principles of stone structures with small elements [39].

The photos used in the figures were taken by the authors, in some cases the materials from Sh. Azatyan's personal archive were used. The drawings of the portals were made by the authors on the basis of A. Sahinyan's, T. Marutyan's, S. Mnatsakanyan's, N. Tokarski's restoration materials, as well as their own measurements. In the case of the portals of Yereruik and Arutch temples, the original measurement drawings made by Sh. Azatyan were used.

The article has been elaborated based on a study of the 51 portals belonging to 20 early medieval Armenian churches. The geography of the study is mainly limited to the central and north-eastern borders of Ayrarat Province of Greater Armenia (Mets Hayk), which includes a significant part of the present-day Republic of Armenia and covers the 4th-8th centuries Dioceses Ayrarat, Arsharunik, Vanand, Bagrevand and the northern part of Basen (Fig. 1). This is due to the historically central importance of the province, as well as the problems of access to the monuments in the current geographical and political situation [10].

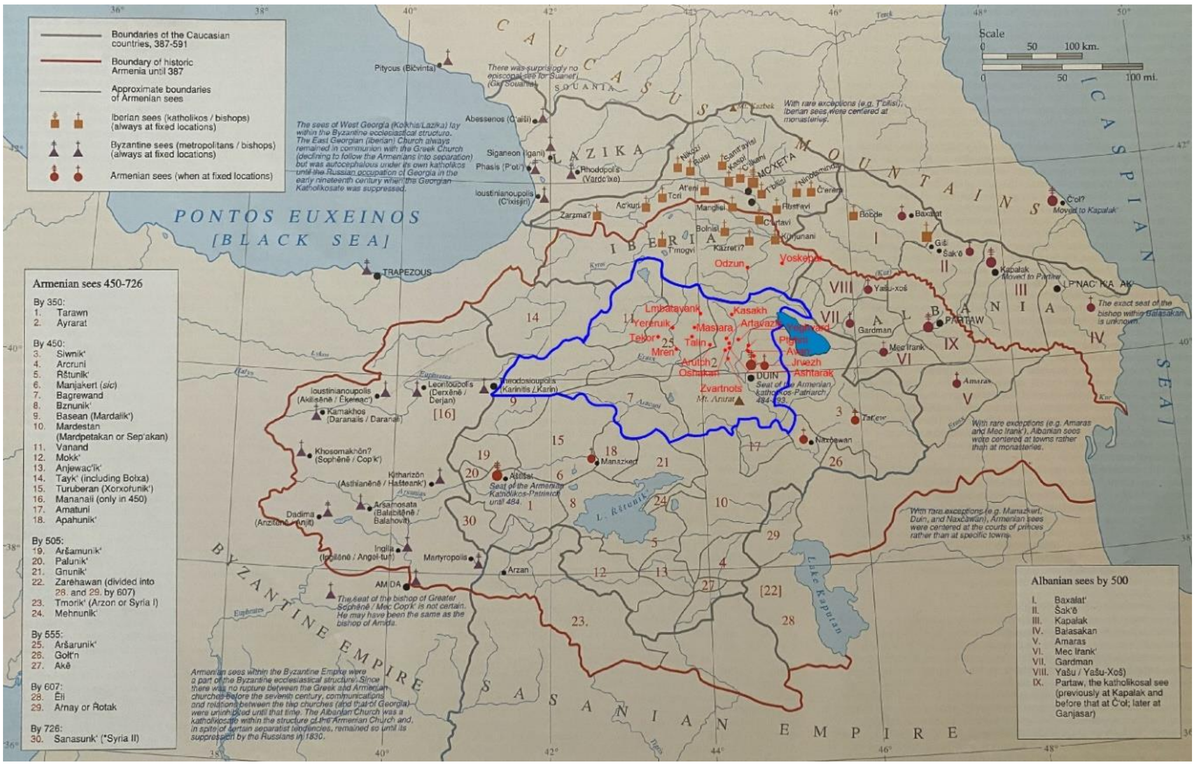

Fig. 1. *Historical Armenia with the location of the Ayrarat Province and the monuments examined (the map: Hewsen RH, Armenia: A Historical Atlas, Chicago, University of Chicago Press, 2001, p.77, Fig. 64, The Development of the Armenian Episcopal Sees; Boundaries of Ayrarat Province: according to State Committee of the Real Property Cadastre of the Government of the Republic of Armenia, National Atlas of Armenia, Yerevan, 2017, p.30-31)*[1]

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Results

The general chronological-typological classification of the portals of Armenian medieval monumental structures shows the formation patterns for the period of the 4th-7th centuries [3]. According to that classification, there are two main types of church portals in 4th-7th centuries (Fig. 2). The first type is the rectangular opening, on both sides of which are placed pylons protruding from the plane of the wall, connected by a horseshoe arch with a pediment covering. This simple but compositionally expressive form of the portal, as a result of the constant development of creative thought and constructive solutions, comes in its numerous varieties in 4th-7th centuries (Kasagh, Yereruik, Ptghni, Avan, Zvartnots, Talin (Cathedral) and other churches). And from the 6th century, another type is also used, which is created by removing the pediment from the portal covering. Such a solution can be seen in the Yeghvard, Mren, Talin (St. Astvatsatsin) and other churches.

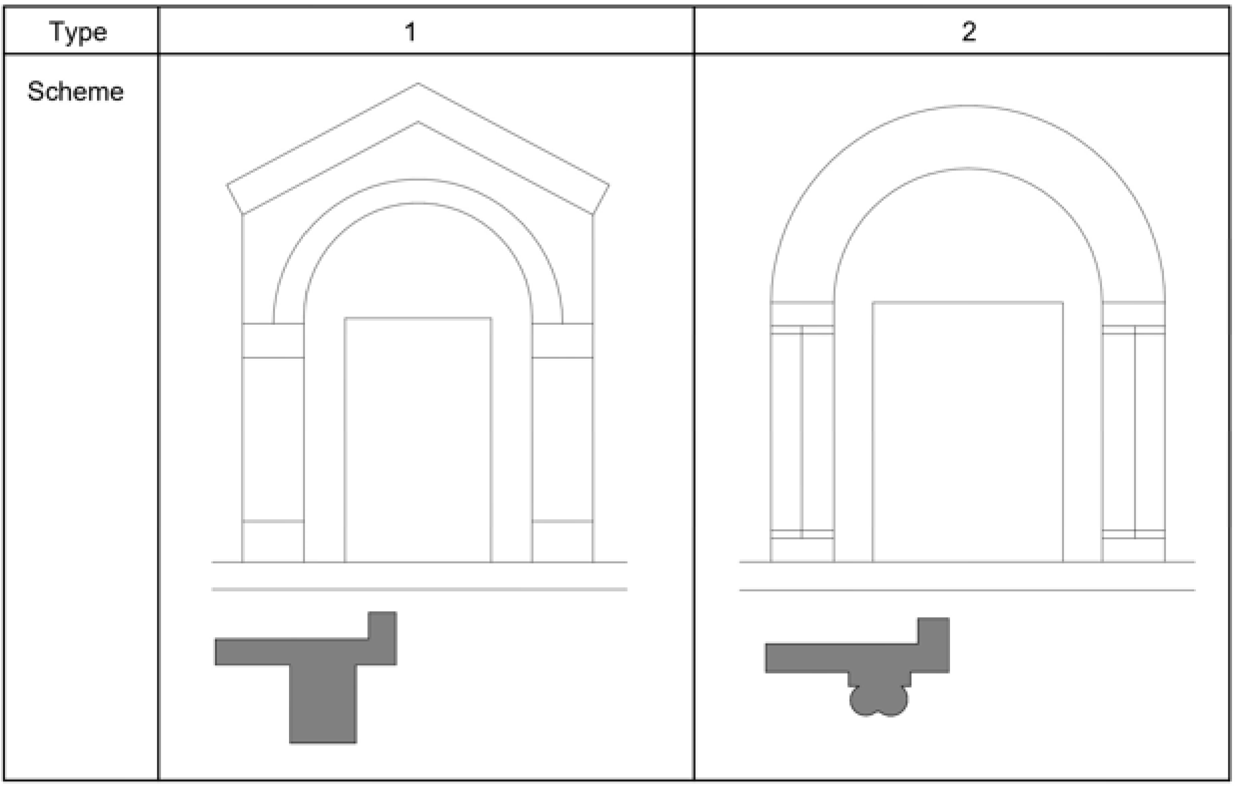


Fig. 2. *The period of 4th-7th centuries of chronological-typological classification of the portals of Armenian medieval churches (according to Azatyan Sh.R., Portals in the Monumental Architecture of Armenia in 4th-14th centuries, Yerevan, Sovetakan Grogh, 1987, appendix, Fig. 42)*

Both of the above-mentioned types of portals of the 4th-7th centuries generally consist of the same two main compositional-structural components. The first is the decorative volume of the portal protruding from the plane of the wall. Having a variety of decoration of the details, it retains the basic compositional elements that make up the volume: bases, pylons, capitals and an arch (with or without a pediment). The second is the part of the wall between the decorative volume and the door opening, within which the covering of the door opening is carried out (Fig. 3).

The issue of the constructive relationship between these two components, as noted in the study dedicated to the Palazzo del Priori portal damages, as a problem of construction mechanics is of great importance from the point of view of the portal structure formation [4]. It is noteworthy that in the vast majority of the portals of this period, as the examination of the examples below shows, the decorative part is built independently and does not participate in the distribution of the load coming from the wall of the structure.

And in this sense, the constructive solution of the door opening covering is of special interest. It has had a significant role in the development of portals, their formation processes, as this is where the decorative and constructive problems of portals are mainly related [28].

Fig. 3. *The compositional-structural system of the portals of 4^{th}-7^{th} centuries*

The second element that forms the portal, the part of the wall, comes in a number of variants, which are the result of the gradual development and transformation of the lintel construction. Despite the development of stone arch and vault constructions in 4^{th}-7^{th} centuries [18], the door opening continues to maintain the rectangular solution. This leads to a peculiar development of the lintel construction. And the rectangular shape of the opening, which is necessary for the installation of the door and its convenient operation, is decisive both for the development of the lintel construction and for the whole portal formation.

The peculiarities of the development of the lintel constructive system can be revealed by observing typical examples.

Beam. The rectangular door opening in the portals of the earliest churches of 4^{th}-5^{th} centuries is covered with a full single-piece stone beam. In general, coverings with such constructive system have been implemented since ancient times, a vivid example of which is the widespread dolmens in the Eurasian continent **-** burial and cult structures [12, 20, 22]. This system has existed in Armenian architecture until the late Middle Ages. However, in parallel with the creation of the large spans cover systems with significant advantages from the constructive and technological point of view, the beam model continues to be applied, especially in the case of small spans, such as portals. In particular, such a constructive solution has the western portal of the Kasagh Basilica, where the decorative part (almost not preserved on the spot, but the excavated fragments were the basis for A. Sahinyan's restoration) has a completely independent solution from a constructive point of view (Fig. 4).

The beam covering the door opening consists of two parts: external (frontal) and internal beams, which together cover the entire thickness of the wall. The width of the door opening is 1500 *mm*. Beams with a height of 750 *mm* with the same 750 *mm* size rest on the side walls of the door opening. The front beam is decorated with ornaments typical of the time.

The northern portal of the Avan Temple, the portals of the Tekor Temple, of Tsiranavor Church in Ashtarak and of St. Astvatsatsin Church in Talin have such a solution (Fig. 5).

It should be noted, however, that in this system, from the point of view of the structure operation, the lintel stone is in a rather unfavorable condition. It bears all the load of that part of the structure wall, as a result of which cutting forces appear in the supporting parts of the beam, the impact of which increases sharply, especially under the influence of earthquakes or ground settlements [4, 30, 32, 29, 11]. The study of the portals of different churches shows that the mentioned defect is discovered by builder-architects quite early, and

attempts to solve the problem are already noticeable in the structures of almost the same period, which are expressed in other approaches to the implementation of the lintel.

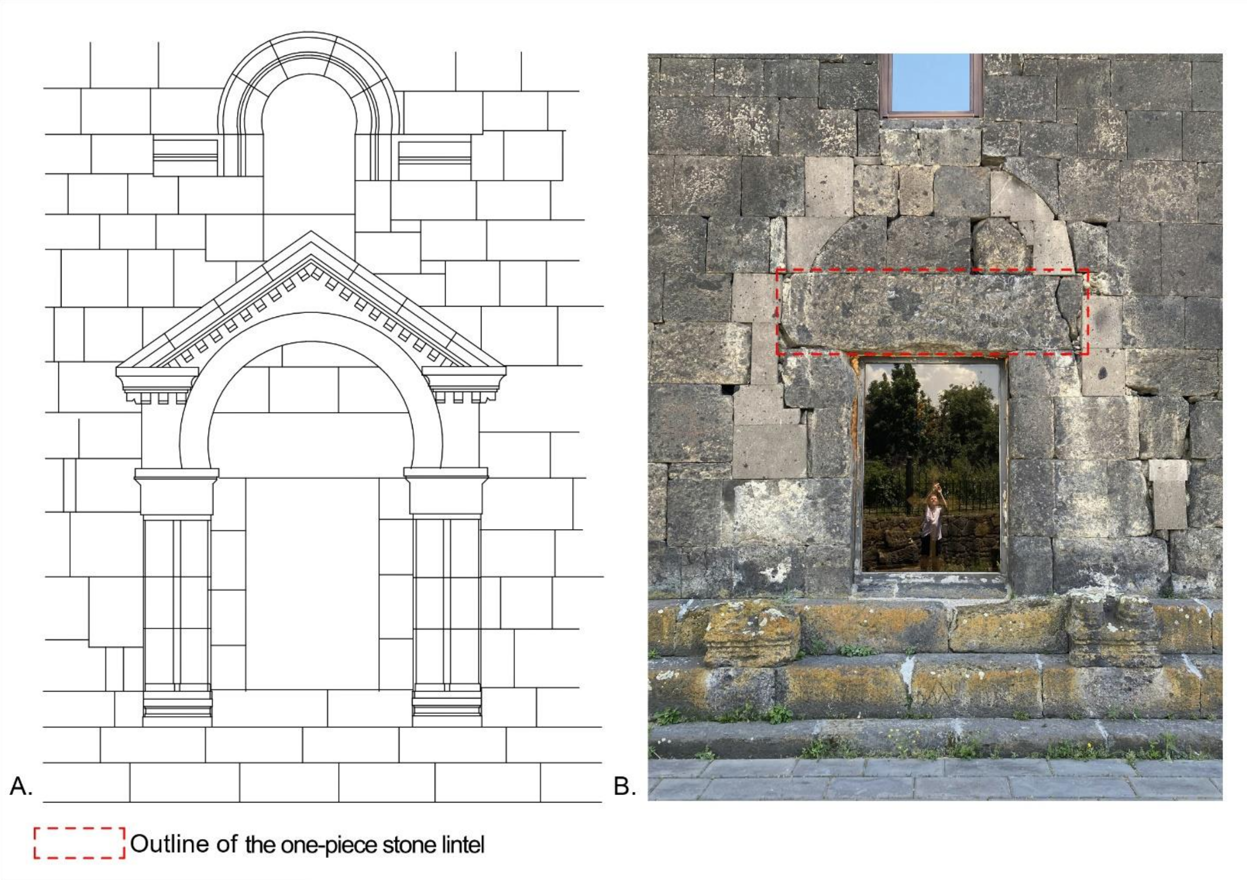


Fig. 4. *The portal on the western facade of the Kasagh Basilica (4th century, Aparan, Aragatsotn Province, Republic of Armenia): A) facade (according to Sahinyan A.A., Architecture of the Kasagh Basilica, Yerevan, Academy of Sciences of the Armenian SSR, 1955, p.149, Fig. 123); B) general view*

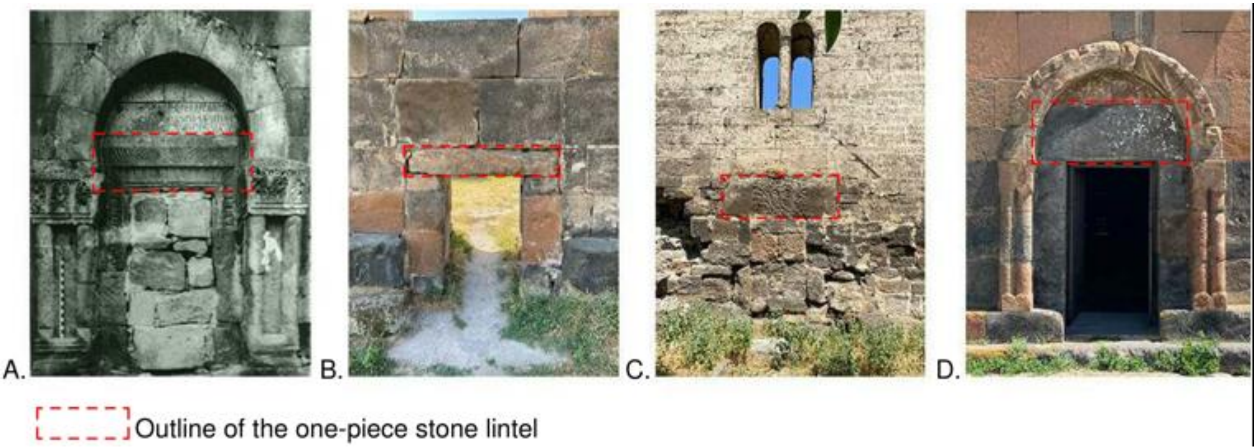
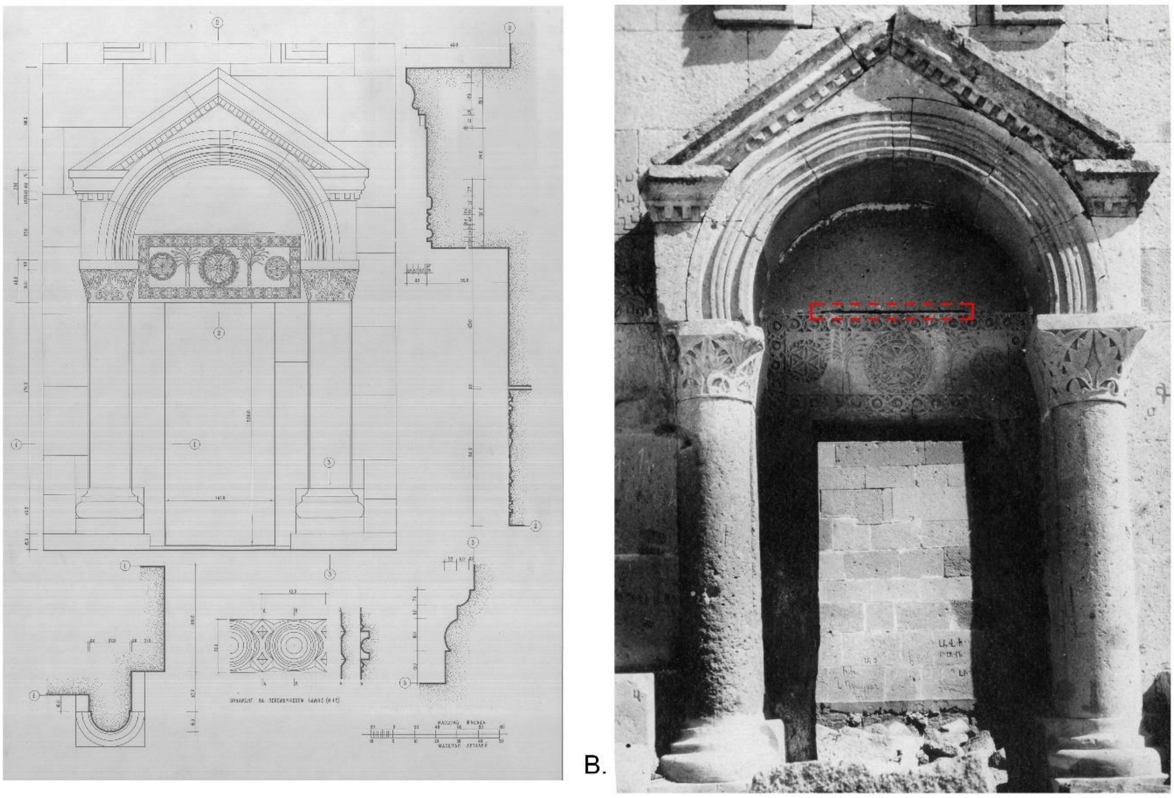


Fig. 5. *A) The portal on the northern facade of the Tekor Temple (5th century, Digor, Kars Province, Republic of Turkey); B) the portal on the northern facade of the Avan Temple (591-602, Yerevan, Republic of Armenia); C) the portal on the western facade of the Tsiranavor Church (5th century, Ashtarak, Aragatsotn Province, Republic of Armenia); D) the portal on the western facade of the St. Astvatsatsin Church (7th century, Talin, Aragatsotn Province, Republic of Armenia)*

Slot. In the 4th-5th centuries, in parallel with the widespread beam covering, another type of door opening covering was created. It is expressed in the system of lintels with unloading slots. The slots are implemented

between the stones of the first and second rows of the lintel, on the stone of the second row. Due to the slots, the lintel stone is released from the directly affecting load, and all the weight coming from the wall is transferred directly from the stone of the second row to the lateral parts of the opening. Here the lintel turns into a self-bearing constructive element that ensures the rectangular opening of the door. Interesting examples from the point of view of the construction and artistic design of this type of covering on the church entrance are the southern and western portals of the Yereruik Temple (Fig. 6).

A.



Slot outline

Fig. 6. *The right side portal on the southern facade of the Yereruik Basilica (5th century, Anipemza, Shirak Province, Republic of Armenia): A) facade (according to Azatyan Sh.R., Portals in the Monumental Architecture of Armenia in 4th -14th centuries, Yerevan, Sovetakan Grogh, 1987, p.58, plate 1, the original drawing used); B) general view*

The opening of the western portal door with a 1560 *mm* span is covered with two large lintel stone blocks – external and internal. The unloading slot is implemented on the second row of blocks, within the whole depth of the wall and has 35 *mm* height. The artistic development of the lintels on the temple portals is closely connected with its construction. In all three portals, the stone of the first row of the lintel, which is separated from the other rows by an unloading slot, is developed in detail. Such a design of developing the lintels ensured the originality of the artistic expression of the portals [21, 8].

However, it should be noted that the unloading slots, while improving the construction of the lintel, do not give a complete solution to it. Covering the doorway with large stone blocks is of technological and construction difficulties and lags behind the development of construction techniques of the time, where the principle of implementation of structures (walls, arches, vaults) by relatively small elements is already widely used [39]. And such an approach to covering the door opening as in Yereruik Temple was later left out of the portal implementation practice and was found in the structural solutions of smaller internal doors of the buildings, as in the case of the 4 internal doors of Avan Temple (Fig. 7). It should also be noted that the decorative part in the portals of Yereruik Temple is also independent and does not take part in the general operation of the wall structure.

Fig. 7. *The inner doors of the Avan Temple (591-602, Yerevan, Republic of Armenia)*

Lunette. The logical continuation of the unloading slot is the lunette applied in the practice of construction of the portals of already 6th-7th centuries. From the engineering point of view, lunette plays the role of the slot. In this case, the constructive principle of releasing the lintel stone from direct load remains unchanged, only the stones of the second row of lintel complicated from the point of view of technology and construction, are replaced with an arch, which also has an advantage in terms of reliability [6]. In particular, an example of the lunette use is the western portal of the Avan Temple (Fig. 8). This example, however, manifests that the lunette solves a partial problem in the construction of the lintel. Although the lunette arch bears the load of the wall, the self-supporting stone of the lintel required for the installation of the rectangular door, which is preserved in this type as well, is in a rather unfavorable condition from the constructive point of view. The span part of the beam is free from bottom and top parts, and at the edges - in the supporting parts, it takes the forces coming from the arch. In the case of large spans and large dimensions of structural elements, a vivid example of which is the western portal of the Avan Temple, damages occur under the influence of cutting forces during seismic tremors [7]. These become the reason for the implementation of an additional wall section in the doorway opening of this portal later (Fig. 8).

Examples of the application of lunette are also found in the portals of the Zvartnots Temple and the Jrvezh Church (Fig. 9). In Zvartnots Temple, which is one of the most important monuments in the field of Armenian church architecture, all 5 portals have similar solution: they all have lunette [17]. It should be noted that according to A. Yeremian's comparative analysis, there were portals with such a solution in St. Hripsime Church (618, Vagharshapat, Republic of Armenia). However, they were later substantially changed and complete facts on original solutions were not preserved [41].

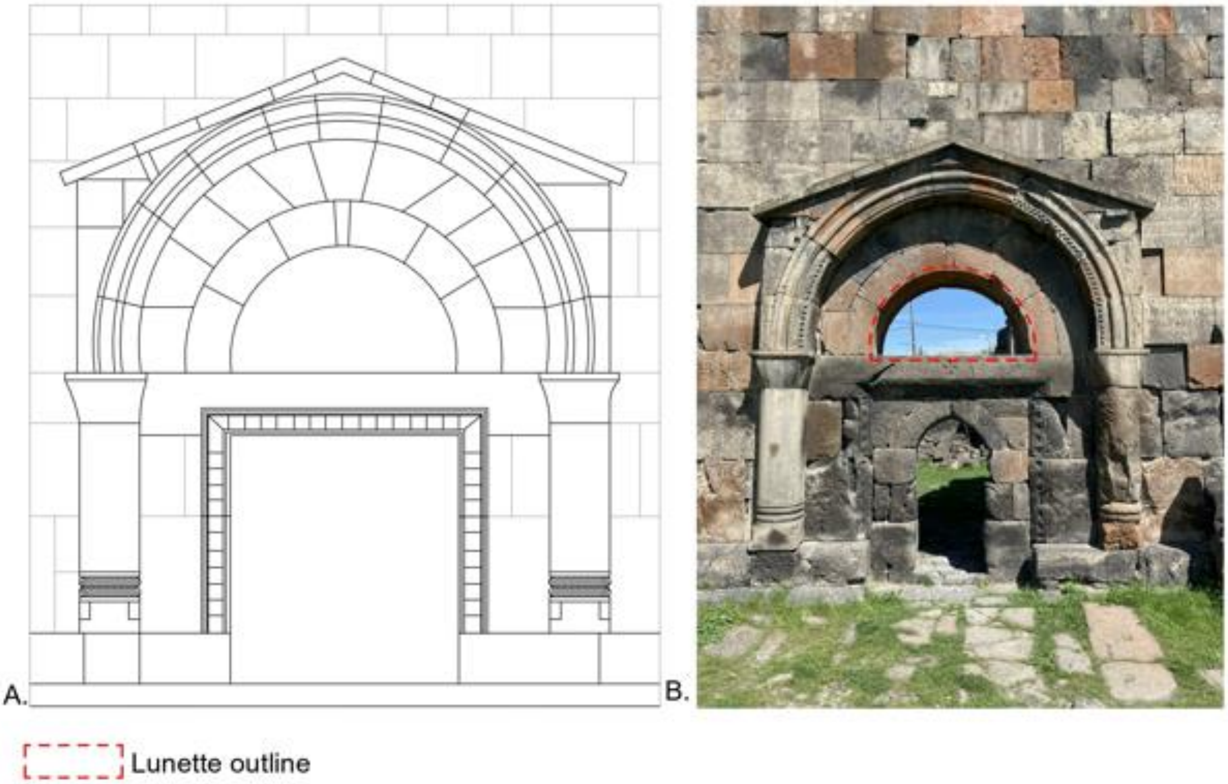


Fig. 8. *The portal on the western facade of the Avan Temple (591-602, Yerevan, Republic of Armenia): A) facade (according to Marutyan T.H., Avan Temple and Similar Monuments, Yerevan, Hayastan, 1976, p.19, Fig. 12); B) general view*

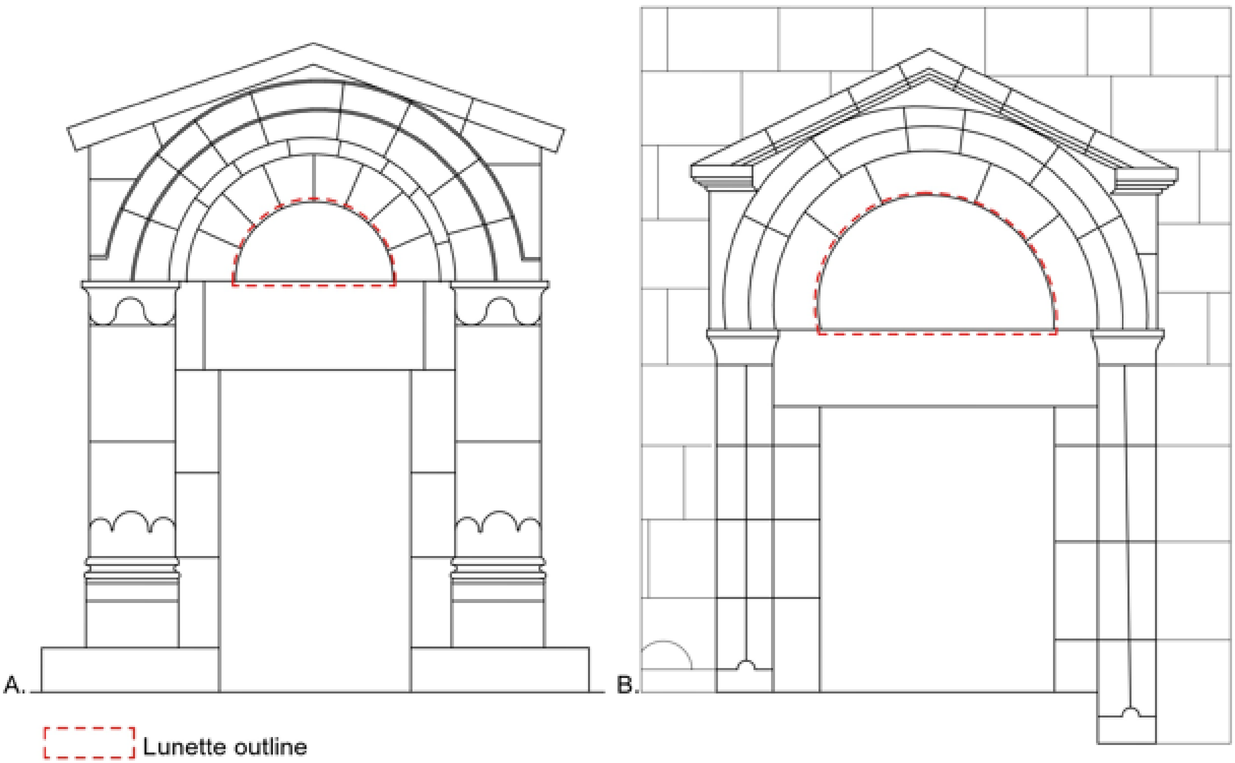


Fig. 9. *A) One of the portals of the Zvartnots Temple (641-661, Zvartnots, Armavir Province, Republic of Armenia: according to Mnatsakanyan SKh, Zvartnots, Moscow, Iskusstvo, 1971, p.29, Fig. 11); B) the portal on the western*

facade of Jrvezh Church (7th century, Jrvezh, Kotayk Province, Republic of Armenia: according to Tokarsky NM, Jrvezh, Yerevan, Academy of Sciences of the Armenian SSR, 1959, p.57, Fig. 20)

Closed Lunette. The application of lunette creates a new type of portal with unique artistic expressiveness. However, this option of covering the door opening does not become widespread as well. It can be assumed that one of the reasons for the rejection of this covering method were the difficulties related to closing the opening in the lunette part.

The problem of closing the lunette opening is evidenced by the fact of implementation of stone masonry in those parts later, as well as the creation of closed lunettes, where the lunette is replaced by a stone block with a circular top part. An example of this is the western portal of the 7th century Mren Temple (Fig. 10).

Fig. 10. *The covering structure of the western portal of the Mren Temple (640, Kars Province, Republic of Turkey)*

This, being an example of a closed lunette, is at the same time quite one of a kind. Here the lintel stone is not rectangular but trapezium-shaped. The width of the stone closing the lunette is smaller than the door opening and is also smaller than the size of the upper zone of the trapeziform lintel. Both of these stones are carved with bas-reliefs with unique religious and secular motifs. The portal on the northern facade of the Temple has the same solution [19].

There are portals with closed lunette also in Lmbatavank. The application of such a solution can also be found in the inner doors of the churches, in particular, in the sacristy entrances of the Ptghni Temple (Fig. 11).

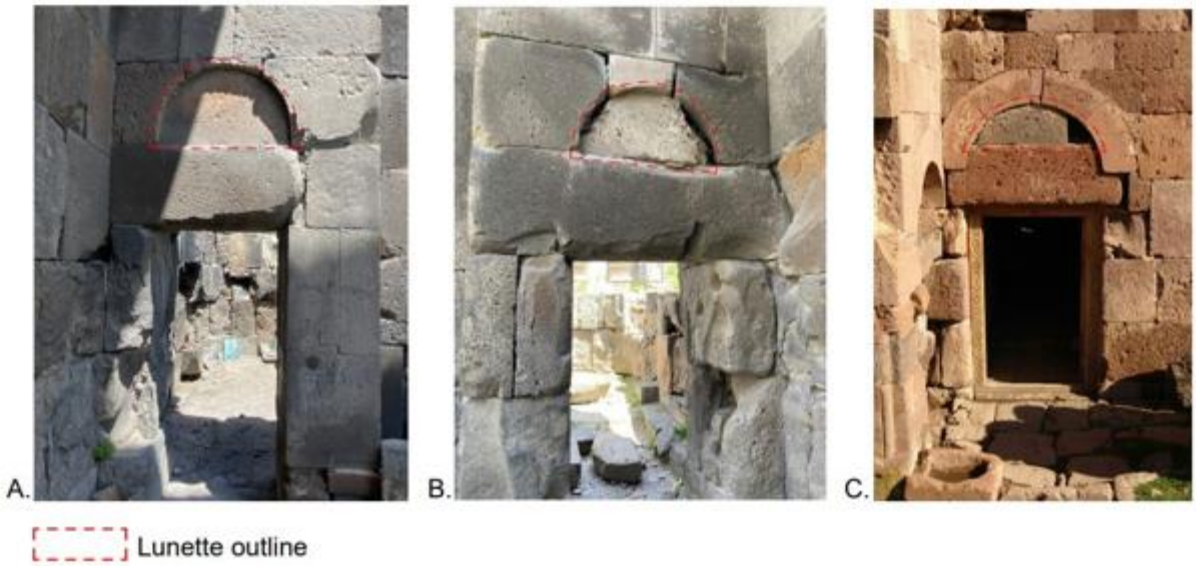


Fig. 11. *A, B) The inner doors of the Ptghni Temple (6^{th} century, Ptghni, Kotayk Province, Republic of Armenia); C) the portal on the western facade of the St. Stepanos Church of Lmbatavank (7^{th} century, Artik, Shirak Province, Republic of Armenia)*

Beam and Arch. Another approach to the lintel unloading is the combination of different solutions in two layers of the wall masonry (external and internal). In this case, the beam carrying the load from the wall is preserved in the external part, and an arch of a size of the door opening is implemented in the internal part. Such a solution can be seen in the portals of the Arutch Temple (Fig. 12). These are implemented in a form of a niche-portico covered by span roof with the appearance of a pediment. This serves both the purpose of protecting from various weather conditions and, apparently, of forming appropriate psychological preparations before entering the temple [1]. In the lintel part, there is a masonry with a semicircular top consisting of three stones, of which the lower beam stone bears the load of the external layer masonry of the wall. The covering of the door opening implemented in a form of a semicircular arch from the inside, which is the direct support of the internal layer masonry of the wall.

Fig. 12. *The portal on the northern facade of the Arutch Temple (666, Arutch, Aragatsotn Province, Republic of Armenia): A) facade (according to Azatyan Sh.R., Portals in the Monumental Architecture of Armenia in 4^{th}-14^{th}*

centuries, Yerevan, Sovetakan Grogh, 1987, p.66, plate 5, the original drawing used); B) general view from the outside; C) general view from the inside

The application of the combined solution is also found in Odzun, Ptghni, Yeghvard, Mastara, Byurakan Artavazik churches (Fig. 13).

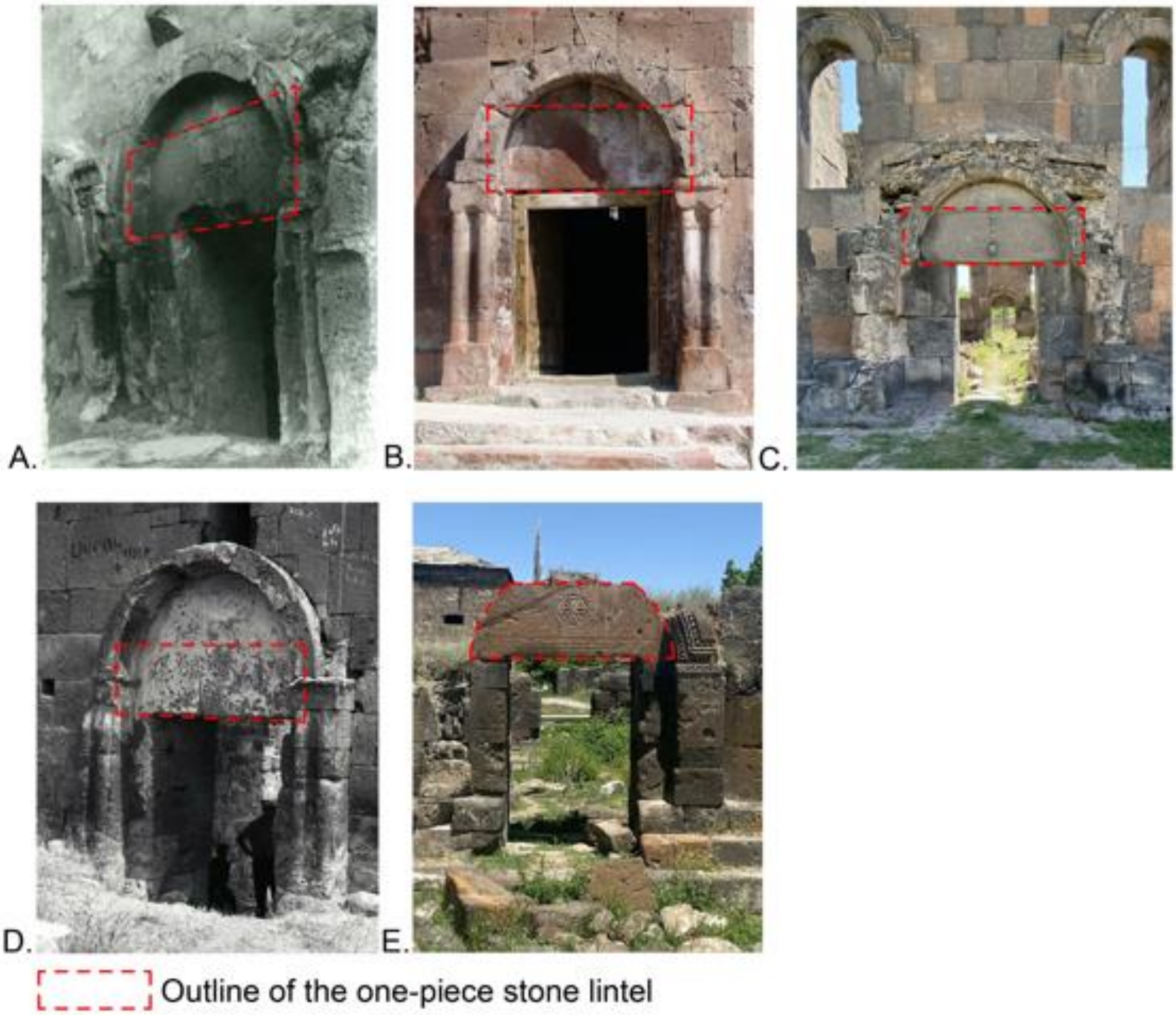


Fig. 13. *A) The portal on the southern facade of the St. Astvatsatsin Church (6th century, Odzun, Lori Province, Republic of Armenia); B) the portal on the western facade of the St. Hovhannes Church (7th century, Mastara, Aragatsotn Province, Republic of Armenia); C) the portal on the northern facade of the Ptghni Temple (6th -7th century, Ptghni, Kotayk Province, Republic of Armenia); D) the portal on the western facade of the Artavazik Church (7th century, Byurakan, Aragatsotn Province, Republic of Armenia); E) the central portal on the southern facade of the Yeghvard Temple (6th century, Yeghvard, Kotayk Province, Republic of Armenia)*

It should be taken into account that the presence of a portico at the Arutch Temple portals, however, does not affect the solutions of the opening covering, and the decorative part of the portal does not bear the load of the external layer masonry of the wall in any way. And the mentioned, in fact, also refers to the portals of all the churches already discussed. Here, the components of the decorative part of the portal - arch, vault, pediment, pylol, base, capital are all inserted in the wall slightly or are placed in front of the wall [33]. They do not directly participate in the transfer of the load of the entire wall height to the ground, and the elements of the upper zone generally rest on the construction of the door opening lintel.

Unloaded Tympanum. A direct consequence of above-mentioned approaches to the closed lunette and the combination of beam and arch in two layers of the wall masonry can be considered the creation of a new type of door opening covering, where the stone of lintel beam as well as the stone closing the lunette are united in one block - the tympanum stone. Such a solution can be seen in the portals of the 7th century churches

Mankanots in Oshakan, Karmravor in Ashtarak, St. Astvatsatsin in Voskepar and Katoghike in Talin (Figures 14, 15).

Fig. 14. *A) The portal on the western facade of the St. Astvatsatsin Church (7th century, Voskepar, Tavush Province, Republic of Armenia); B) the portal on the western facade of the Mankanots Church (7th century, Oshakan, Aragatsotn Province, Republic of Armenia); C) the portal on the western facade of the Karmravor Church (7th century, Ashtarak, Aragatsotn Province, Republic of Armenia)*

Both portals of St. Astvatsatsin Church in Voskepar have this solution. Here the decorative volume is completely independent of the tympanum. The tympanum of the western portal has cracked in its entire height under the weight of the upper part of the wall [16]. Both portals have an arch of a size of the door opening on the inside (Fig. 14a). The tympanum of the portal of the Mankanots Church in Oshakan has the same solution (Fig. 14b).

In the tympanum of the portal of the Karmravor Church in Ashtarak, the ratio of the tympanum and the decorative section is more obvious and clear, as the decorative part of the portal is missing, due to which the constructive solutions of the door opening part of the wall are visible. The tympanum has wings at the edges, on which the arched stones placed above the tympanum rest. There is an arch of a size of the doorway from the inside (Fig. 14c).

In the Talin Cathedral portals, which have a portico solution like the Arutch Temple ones, the door opening covering is externally implemented with the above-mentioned tympanum stone block, and internally, as in the Arutch Temple, with an arch. The tympanum stone, which has wings in the bottom part, is developed from two sides, externally for the facade and internally for the interior. Here the internal arch is semicircular, the height is less than the height of the tympanum and acts as a support for it (Fig. 15).

The tympanum created by the mechanical union of two constructive elements (the lintel beam and the stone closing the lunette) had partial defects. The first is that rectangular wings inertially left from the lintel are of no constructive significance. The second is that these wings hinder the complete implementation of the arch built above the tympanum, which in turn prevents the complete unloading of the tympanum. Tympanum stones of the Talin Cathedral typically have cracks along the entire height of the rectangular wings (Fig. 15).

It should be taken into account that the covering solution for the Talin Cathedral portals differs significantly from the versions of Mankanots in Oshakan, Karmravor in Ashtarak and St. Astvatsastin in Voskepar. Here, the stones of the arch built above the tympanum are exactly the stones of the decorative part, which are no longer inserted in the wall, but go deep inside, taking the load of the external layer of the wall (Fig. 15).

This allows us to prove that the creation of this type of door opening covering is a turning point in the formation of portals. If before that the decorative volume and the part of the wall that made up the covering of

the door opening were independent from the structural and constructive point of view, here the portal arch, while preserving the decorative function, at the same time bears the load of that part of the wall and becomes a constructive component, leaving to the tympanum stone only the functions of insulation and giving a rectangular form to the door opening.

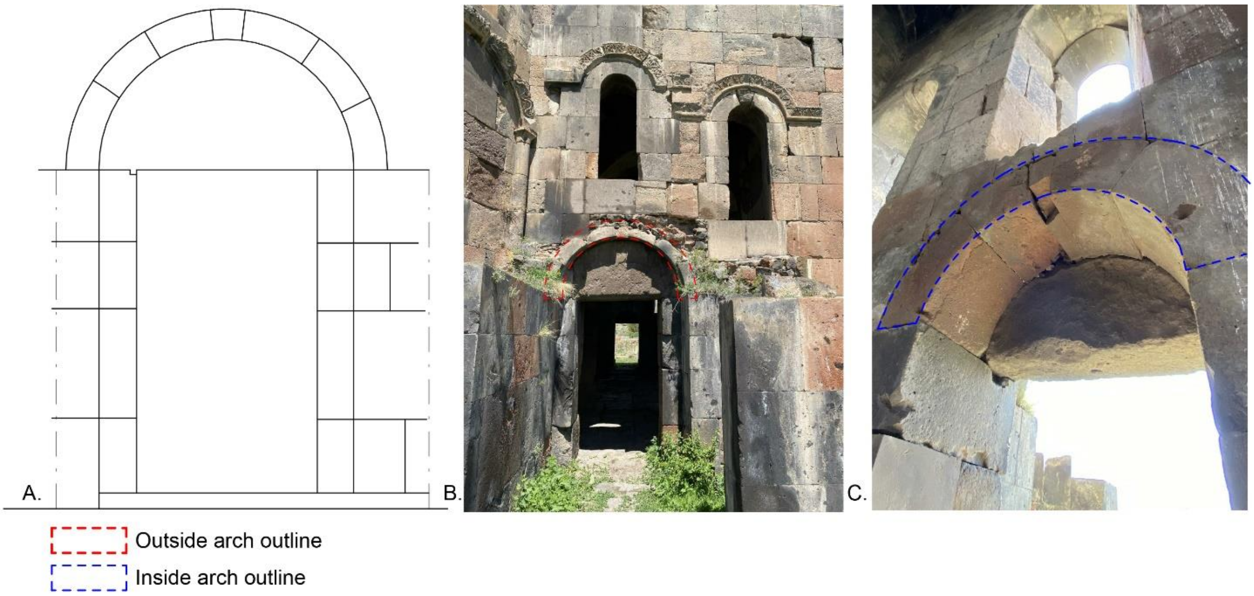


Fig. 15. *The right side portal on the western facade of the Talin Cathedral (7th century, Talin, Aragatsotn Province, Republic of Armenia): A) facade; B) general view from the outside; C) general view from the inside*

It is noteworthy that this new type of tympanum which gives new opportunities of artistic design and is more reasonable from the constructive point of view, undergoing some improvements (the wings left from the lintel beam are removed, which allows the implementation of a complete load-bearing arch, in the supporting parts the tympanum corners intersect at 45 degrees, as a result of which the cutting forces at the edges are eliminated), becomes primary in the Armenian medieval monumental architecture and greatly widespread in later centuries [40, 13, 37, 3, 34, 24].

Summary and Conclusion

The analysis of the development of the portal architecture of the Armenian churches of the 4th-7th centuries based on the methods of theoretical research, comparison, classification and generalization allows revealing some regularities of the interaction of constructive and artistic solutions in the process of portal formation. In particular, from the point of view of the transformation and development of the lintel construction, it is possible to make a certain typological classification of the portals of 4th-7th centuries Armenian churches (Fig. 16).

The classification shows that although from the point of view of the lintel structure there is a certain evolution in the typological transformation process of the portals, it does not have a clear, definite direction. Originating from the ancient beam type, it has in parallel led formation of new types expressing the attempts to unload the lintel, which, however, did not exclude the use of the beam type. At the same time, as a result of this multilateral process, the preconditions for the structural-artistic unity of the architecture of the portals were formed, which became primary in the further Armenian medieval architecture.

In particular, the classification makes it obvious that significant changes and differences in the lintel construction appear in the second element that forms the portal - in the wall part and depend on the constructive solution of the lintel. While following the development of the forms in the tympanum part, the desire to find a more structurally stable, reliable system for covering keeping the rectangular form of the doorway becomes obvious. As a result, there are changes in the structural role of the two main compositional and structural components of the portal - the decorative and wall parts. The decorative part is included in the constructive

system of the structure wall, and the part of the portal wall covering the doorway becomes a self-supporting tympanum, free from the load of the structure wall.

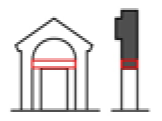
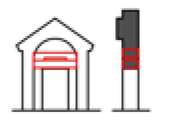
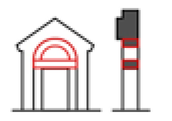
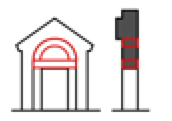
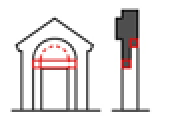
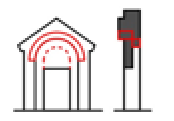
Type	Beam	Lintel unloading attempts				Unloaded tympanum
		Slot	Lunette	Closed lunette	Beam and arch	
Scheme (facade and section)						
4th century	3 portals of the Kasagh Basilica	—	—	—	—	—
5th century	4 portals of the Tekor Temple, 2 portals of the Tsiranavor Church in Ashtarak	3 portals of the Yereruik Basilica	—	—	—	—
6th century	Northern portal of the Avan Temple	—	Western portal of the Avan Temple	—	3 portals of the St. Astvatsatsin Church in Odzun, 3 portals of the Ptghni Temple, 5 portals of the Yeghvard Temple	—
7th century	The portal of the St. Astvatsatsin Church in Talin	—	5 portals of the Zvartnots Temple, western portal of the Church in Jrvezh	Western and northern portals of the Mren Temple, 2 portals of the St. Stepanos Church of Lmbatavank	2 portals of the St. Hovhannes Church in Mastara, 3 portals of the Arutch Temple, the portal of the Artavazik Church in Byurakan	5 portals of the Talin Cathedral, 2 portals of the St. Astvatsatsin Church in Voskepar, the portal of the Mankanots Church in Oshakan, the portal of the Karmravor Church in Ashtarak

Fig. 16. *Typological classification of the portals of Armenian churches in 4th-7th centuries according to the transformation and development of the lintel construction*

Thus, it can be stated that the portal, which was a form of a decorative design of the entrance in the architecture of the 4th-7th centuries Armenian churches, acquires a new artistic expression at each stage of the lintel construction evolution, and, taking the functions of bearing the covering as well as the applying hardness to a wall weakened by the doorway, becomes a unified constructive-artistic element.

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The Dependence of Durability and Deformation of Elements of Soil Cement Composite with Carbonate White Soil Mixture on Age

The results of an experimental study of changes in strength and deformability during the period of time subjected to short-term loading of elements made of a soil-cement composite based on white soil (belozems) of carbonate composition are discussed.

Research was carried out in accordance with current standards, as well as a well-known method that has been repeatedly tested earlier.

To assess the experimentally established data, the results of similar studies by other authors, carried out applying elements from soil-cement based on clay soils, as well as from lightweight concrete on lithoid pumice (volcanic rock), are also presented.

On the basis of the comparative analyzes of the experimentally established data, conclusions are formulated. The consideration of those may be useful both for the estimation of optimal schedules of the construction of buildings from a soil-cement composite, and for the assessment of their stress-strain state.

Keywords: *Soil cement, white soil (belozem), carbonate mixture, durability, deformation module of deformations, coefficient of transversal deformations, stress-strain state.*

Introduction

It is common knowledge that in many composite construction materials and soil cement is not exception. Cement is used as a connective component as a result, through merging and solidification composites are formed on its base.

There is scarce research (1,2) dedicated to the study of impact of time in regard to changes in the durability of soil cement. According to the abovementioned unique studies, right after its making during some time (time span lasts from 28 up to 60 days) the durability index constantly increases, leading to the compression of soil cement [1]. It should be noted that the speed of increase in the soil cement durability at the initial stages of observation (during the first 28 days after the production of the material) turns out to be quite essential, while later on it gradually fades[1].

The given research focuses on the results of studies, concerning the dynamics of changes that take place over time in the durability and resistance to deformation observed in the short-term loading of elements made of soil cement composite and white soil (belozem), carbonate mixture.

Materials and Methods

The objects of the research were test specimens - cylindrical elements made of soil cement composite with carbonate, white soil mixture:

- the diameter d = 5.0*cm*, height h = 5.0*cm* for the identification of durability limits of the material under compression,
- diameter d = 5.0*cm* and height h = 20*cm* for the identification of deformation characteristics of soil cement elements in short-term loads [2].

[1] GOST 18105-2018. Concretes. Rules for control and assessment of strength, Moscow, Standardinform, 2019.

For the production of soil cement composite the following components were used:

- white soil (structurally unstable soil on a territory. In areas where white soil extends it is not recommended to construct civil or industrial buildings) taken from the areas that are neighboring the Institute of Physics, situated in the residential district of Ajapnyak, Yerevan,
- portland cement of 40 *MPA* produced by Ararat cement factory (Armenia),
- tap water.

Based on the analysis of chemical and saline composition of white soil sifted through sieve N 2 it was ascertained that the white soil used in soil cement was presented through powder-like sandy loams [3].

The characteristics of the components used in the production of soil cement composite are highlighted in Table 1.

Table 1. The characteristics of the components used in the production of soil cement composite, its mixture and some data

Characteristics of filling aggregate				Characteristics of binding component	
Item		Admitted fineness, *mm*	Apparent density, t/m^3	Item	Compression strength, *MPa*
White soil of carbonate mixture		2.0	1.410	Portland cement	40.0
Composition soil cement					
Consumption of components for 1m^3 soil cement, *kg*			W/C	Density at max. condensed condition, t/m^3	Strength at the age of 90 days, *MPA*
cement	white soil	water	1.787	1.921	11.2
141.0	1269.0	252.4			

It should be noted that the magnitude of maximum density in the soil cement skeleton - ρ_{dmax} defined by a standard method of compression, using the Proctor Compaction Test Equipment constitutes 1.61gr/cm^3. The value of solidity observed in the skeleton of freshly laid soil cement, ρ_d on average constitutes 1.58gr/cm^3 which is within the limit of admissible magnitude deviation in the given characteristics ($\rho_d = 0.983\ \rho_{dmax}$).

The production of various cylindrical specimens of soil cement composite was made via direct pressing under normal pressure of 4.8-5.3 *MPA*. This secures the production of an ultimately solid material [3].

Some of the produced cylindrical specimens made for the experiment were designed for defining the limits of durability of the soil cement composite as well as for singling out deformation characteristics of the material at different ages (time countdown after their production) were taken out of metallic mould 7 days after their making, later on they were preserved in humid sawdust for another 7 days. After that up until the beginning of experiments the specimens were left at the laboratory.

The measurement of durability limits observed in soil cement composite was made at the ages (τ) of 14 days, 28 days and 90 days, while the identification of deformation characteristics of the material was carried out at the ages of 28 days and 90 days.

Other cylindrical test specimens d = 5.0*cm* and h = 5.0*cm* were designated for studying the impact of air and humidity conditions on the growing durability of soil cement composite. Data on the methods used in these studies will be introduced below.

In the period of carrying out all the studies the average temperature at the laboratory was 23°C and average humidity - 57% (the conditions were close to those observed in natural environment characterized by levels of low humidity) [4].

The measurement of durability levels of pressure on the soil cement composite (using specimens d = 5.0*cm* and h = 5.0*cm*) was made at an average speed of the experimental equipment, displacing functional gears 3*mm* per minute.

The identification of deformation characteristics of soil cement (using the cylindrical specimens d = 5.0*cm* and h = 20.0*cm*) was made according to the method [5] which will be described below.

The loading of test specimens was fulfilled in a stepped way, each step corresponding to $0.1R$ (R is the resistance magnitude of specimen to destruction) of the durability of cylinders under each step was just sufficient to register the data from micron indicators, measuring the longitudinal and transversal deformations. At the level of pressing load corresponding to $0.8R$ the clock indicators measuring deformations were taken and the specimens were destroyed.

The empirical data on the longitudinal (ε) and transversal (ε) deformations of soil cement components were approximated by dependencies [5]:

$$\varepsilon_{\text{пр}} = \frac{a_1 \frac{\sigma}{R}}{1 - b_1 \frac{\sigma}{R}} \quad , \tag{1}$$

$$\varepsilon_{\text{поп}} = \frac{a_2 \frac{\sigma}{R}}{1 - b_2 \frac{\sigma}{R}} \quad . \tag{2}$$

The magnitude, concerning the module of deformation Ē and the coefficient of the transversal deformations $\bar{\upsilon}$ (an analogue of Poisson's coefficient of transverse strain in solid bodies) at different levels of pressure were defined according to the dependencies:

$$\overline{E} = \overline{E}_0 \left(1 - b_1 \frac{\sigma}{R}\right)^2, \tag{3}$$

$$\overline{\upsilon} = \frac{\varepsilon_{\text{поп}}}{\varepsilon_{\text{пр}}} = \frac{a_2 (R - b_1 \sigma)}{a_1 (R - b_2 \sigma)} \, . \tag{4}$$

In the formulas (1)-(4) a_1, b_1, a_2, b_2 are the parameters of approximation of the experimental data of deformations $\overline{E_0} = R/a$ illustrated in (3), the initial module of deformations of the material.

The number of test specimens designed for determining the level of durability of the soil cement at different ages was 4-6 while for identifying the deformation characteristics of the material 3-4 specimens were used. In the case of measuring the deformations of transversal specimens with d = 0.5*cm* and h = 2.0*cm* the maximum dispersion of the magnitude of the same measured characteristics in relation to their average arithmetic value was observed. The aforementioned specimens were tested at the age of 90 days since their making +7.1% and -6.2%.

Discussion of the obtained data

In Figures 1, 2 the data obtained through experiment are introduced (they are highlighted with markers) along with the curves approximating these data which describe the time changes observed in the absolute and relative values of durability and average density of soil cement composite with white soil carbonate mixture. In order to carry out a comparative analysis, the data, concerning the impact of age on the durability of soil cement based on clay soil were also included in work[2].

It is worth mentioning that the objects of the research[3] were cylindrical specimens with the diameter of 5.0*cm* and height of 12.5*cm*, having been made of clay soil (loamy soil – sandy loam) taken from the territory neighboring the city of Ekyabadan in the region of Iran[4]. The outlay of cement 40 *MPa* in dry mass of the soil cement made 7% while the composition of soil constituted 93%. The cylindrical specimens used for the experiment after being made were taken out of moulds and left in common laboratory conditions.

After their production, the specimens underwent a short-term testing for the sake of identifying the durability of the soil cement under pressing in 7 days, 14 days, 21 days, 28 days and 60 days.

As a result of short-term test, we managed to confirm that with the increase of the age τ from 14 days to 90 days soil cement with the base of white soil and carbonate mixture initially underwent a monotonous growth at a high speed which later gradually decreased (Fig. 1a, curve 1). According to the data introduced in Fig. 2

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[2] GOST 18105-2018. Concretes. Rules for control and assessment of strength, Moscow, Standardinform, 2019.
[3] Ibid.
[4] Ibid.

(curve 1) the magnitude of the durability growth indicated above at the age of 28 days and 90 days makes correspondingly, more than 31% and about 40% compared with the initial value of the given characteristics defined at the age of 14 days.

After the experiment specimens were left in the lab, the observed loss of humidity during the time mentioned above (28 days and 90 days) compared with its initial value made 1.6% and 8.4% (Fig. 1b, curve 1).

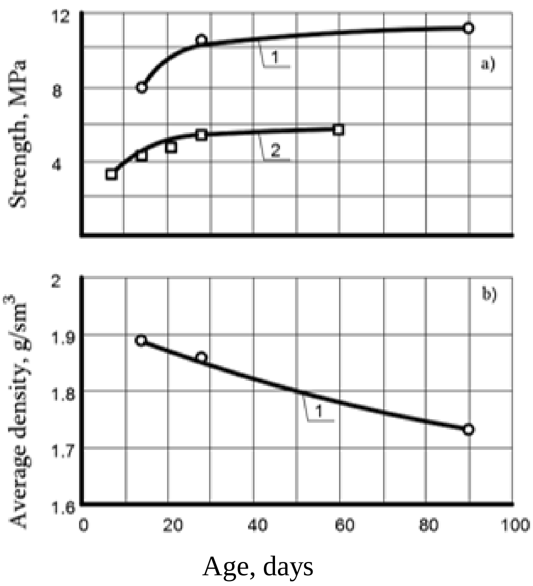
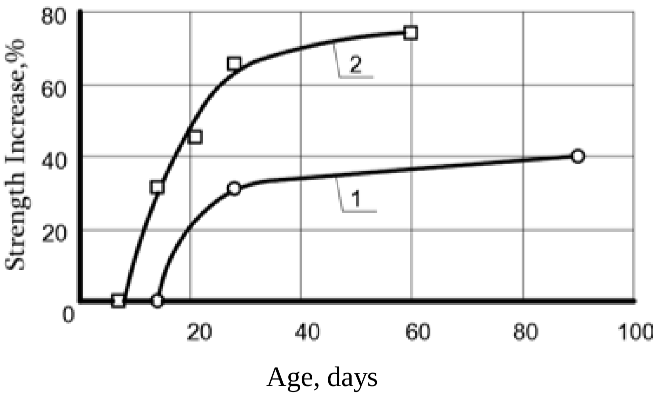


Fig. 1. *Curves of the dependence of the strength value (a, curve 1), the average density (b, curve 1) of the soil cement based on white soils of carbonate composition and the strength (a, curve 2) of the soil cement based on clay soils on the age of the material*

Practically, a similar picture is observed in the research, concerning the impact of time on the levels of durability of soil cement based on the clay soil (Fig. 1a, curve 2). In this case the durability growth of the material at the age of 14 days, 21 days, 28 days and 60 days compared with the initial magnitude of the given characteristics (confirmed at the age of 7 days) approximately makes 31.3%, 45.4%, 65.6%, 74.2% respectively (Fig. 2, curve 2).

It has been mentioned above that the magnitude of the durability growth of soil cement based on the white soil carbonate mixture at the age of 90 days compared with its value of durability at the age of 14 days approximately makes 40%. While according to the data introduced in research[5], the difference between durability value of soil cement based on clay soil identified at the age of 60 days and 14 days approximately constitutes 42.9% (Fig. 2) which, as it can be noticed, slightly increases the value of analogous characteristics identified in soil cement based on white soil and carbonate mixture.



[5] GOST 18105-2018. Concretes. Rules for control and assessment of strength, Moscow, Standardinform, 2019.

Fig. 2. *Curves of the dependence of the strength increase in comparison with its initial value of soil cement based on white soils of carbonate composition (curve 1) and clay soils (curve 2) on the age of the material*

It has already been stated that before doing short-term tests the experiment specimens at different ages in the abovementioned second test right after their making were preserved in the laboratory, whereas in the first test the specimens were taken out of metallic moulds in 7 days and placed in humid sawdust being kept for 7 days thence, the latter were left in the laboratory. This testifies to the fact that there is a great probability that the degree of humidity for 14-day-old experiment specimens of soil cement based on white soil carbonate mixture is relatively greater than the humidity extent observed in specimens of soil cement based on clay soil (we weren't able to find corresponding data, concerning the test on the second type of soil cement mentioned above)[6].

It should be stated that the value of average density of soil cement based on white soil carbonate mixture constitutes 1.57-159*gr/cm*^{3} (see above) while the value of the soil cement based on clay soil approximately makes 1.61*gr/cm*^{3} [7]. It should also be noted that for the production of the abovementioned two types of soil cement the Portland cement has been used as a connecting component, having equal durability under pressure - 40 *MPa*. Consequently, it is permissible to make a comparison between the indices of the same physic-mechanical characteristics common to those types of soil cement.

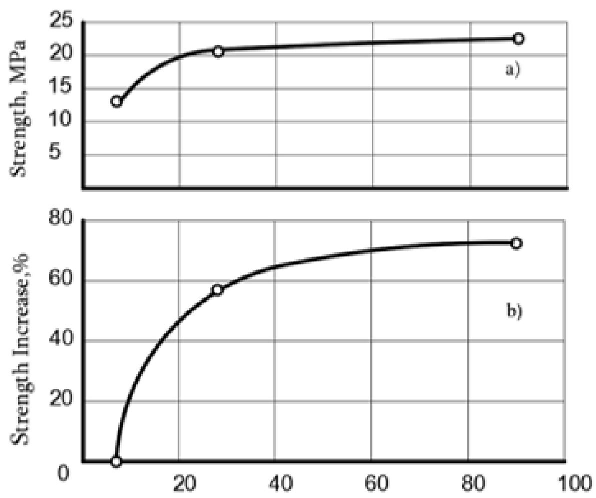
In the work [6] it was stated that for the evaluation of defined mechanical characteristics of soil cement composite it is appropriate to use the corresponding data on other construction materials where the connective component of soil cement is taken into account e.g. for light cement concrete.

According to the abovementioned, in order to make corresponding comparisons we are going to use the data obtained in the research[8], concerning the durability dependence of lightweight lithoid pumice concrete (volcanic mountain type) on the age.

In these studies the cubical specimens being of 15.0*cm*, the body mass 1:1, 54:2, 4, w/c ratio = 0.95 made of lithoid pumice concrete composite were the object for examination. Portland cement of 40*MPa* produced by Ararat cement factory (Armenia) was used.

The cubical specimens made of lithoid pumice concrete right after being taken out of moulds were left in the laboratory where the average temperature was 22°C and the relative humidity - 65%.

The work[9] introduced data that was obtained via experiments, concerning the absolute durability value of lithoid pumice concrete. Based on the durability value of lithoid pumice concrete the growth of the given characteristics acquired throughout time was measured as compared with the durability of the material established at the age of 7 days. We consider it to be appropriate to highlight the latter with curves in Fig. 3.



[6] GOST 18105-2018. Concretes. Rules for control and assessment of strength, Moscow, Standardinform, 2019.
[7] Ibid.
[8] SNiP 2.03.01-84*. Betonnye i zhelezobetonnye konstrukcii, Moscow, 1989, p.77.
[9] Ibid.

Age, days

Fig. 3. *Curves of the dependence of strength and its increase in time in comparison with the initial value of lithold pumice concrete on age*

We can see from the data illustrated in Fig. 3.a, that over time the durability value of lithoid pumice concrete, being under pressure at first increases at a high speed (up to the age of 28 days) and later fades gradually.

The comparison of the data introduced in Fig. 3.b shows that the magnitude of durability growth of lithoid pumice concrete observed at the age of 90 days compared with the magnitude of the given characteristics defined at the age 14 days (in the research this age will be further referred to as initial) constitutes about 42% which slightly exceeds the magnitude of the analogous index established for the soil cement based on the white soil carbonate mixture about 40% (Fig. 2, curve 1). It should be noted that in the dry mass of these two construction materials Portland cement 40 MPa produced by Ararat cement factory (Armenia) has been used as a connective component.

It has been mentioned above that such studies were carried out for investigating the impact of air and humidity on the growing durability of soil cement composite based on white soil carbonate mixture. These studies were carried out with the implementation of method described below.

Some of the cylindrical specimens of the experiment with d = 5.0*cm* and h = 5.0*cm* being taken out of metallic forms after 7 days since their making were immediately put into a package filled with wet sawdust. Further maturing of the experiment specimens took place in the following conditions:

1. Some of the specimens of the experiment after having remained in wet sawdust for 28 days were taken out of the aforementioned package and kept in the laboratory up until the experiment was made.
2. The rest of the specimens were kept in the wet sawdust up until the experiment was made.

The tests of all the aforementioned specimens were done in 180 days after their production. The magnitude of average density and maximum density common to the soil cement skeleton were defined.

The results of the measures are introduced in Table 2.

Table 2

Conditions of ageing of soil cement	Indices of physic-mechanical characteristics of soil cement		
	Density of skeleton, *gr/cm*^{3}	Average density, *gr/cm*^{3}	Durability under pressure, *MPa*
I	1.59	1.675	11.8
II		1.927	13.9

According to the data in Table 2, the maturing in the highly humid environment (in this case in wet sawdust) turns out to have a favorable impact on the growing durability of the soil cement in the course of time.

It should be mentioned that a phenomenon similar to the one illustrated above has been observed earlier in another construction material based on the Portland cement i.e. cement lithoid pumice concrete [7].

Cubes of side sizes 10.0*cm* served as experiment specimens for the studies. The cubes were made of cement lithoid pumice concrete based on the mass 1:1, 513:2, 368, w/c ratio = 0.88. The Portland cement with durability under pressure makes 38 *MPa* produced by Ararat cement factory.

According to the data introduced in the work [7] the durability magnitude of cubic specimens made of cement lithoid pumice concrete made 20.6 *MPa* at the age of 14 days, while after their making for 63 months the latter were kept in favorable conditions (they were preserved in hydro-isolated state), having a positive impact on growing durability. In these conditions their durability constituted 42.2 *MPa*.

Let us discuss the results of the studies on the deformation characteristics of the short-term load of soil cement based on the mixture of white soil carbonate where cylindrical specimens of d = 5.0*cm* and h = 20.0*cm* were used. It should be stated that the resistance magnitude of these specimens under pressure during their destruction in case of τ = 28 days made R = 7.4 *MPa* and in case τ = 90 days the resistance constituted 8.0 *MPa*.

In Fig. 4 the empirical data on ε for longitudinal and ε for transversal deformation specimens are shown with markers along with the curves approximating these data constructed according to the dependencies (1) and (2) respectively. For the approximation of the coefficients the following were taken into account:

in case of τ = 28 days - a_1 = 180.0, b_1 = 0.28, a_2 = 24.0, b_2 = 0.60,
in case of τ = 90 days - a_1 = 138.0, b_1 = 0.53, a_2 = 30.0, b_2 = 0.60.

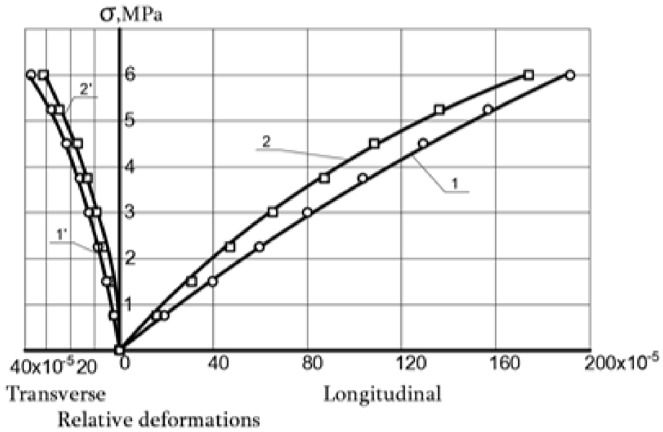


Fig. 4. *Deformation curves of soil cement samples based on white soils of carbonate composition (τ = 28 days - curves 1 and 1', τ = 90 days - curves 2 and 2')*

According to the data in Fig. 4 it can be stated that it is acceptable to use dependencies 1 and 2 for making the approximation of empirical data of the deformations. From the data in the same Figure it can also be noticed that the specimens of τ = 90 days being tested show greater resistance to deformation both in longitudinal and transversal directions compared to their counterparts of τ = 28 days.

From the data illustrated in Fig. 5 it can be concluded that a similar tendency mentioned above is observed in the coefficients of transversal deformations υ of soil cement specimens. From the data of the same Figure highlighted with markers it follows that the results obtained via experiments are satisfactorily described through approximating curves based on the dependencies (4).

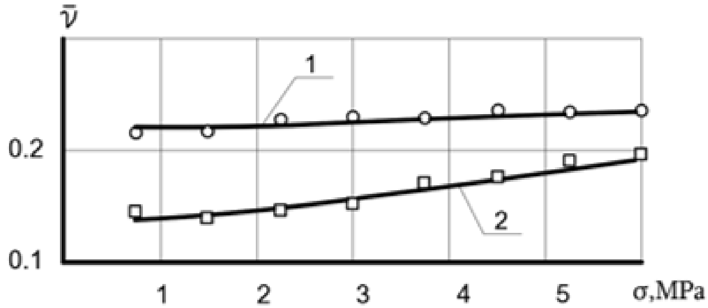


Fig. 5. *Curves of the dependence of the coefficient of transverse deformations of soil cement samples on strength (τ = 28 days - curve 1, τ = 90 days - curves 2)*

The value (calculated on the basis of dependency) of the tangent module of deformation Ē at different levels of pressing stress σ is brought in Table 3.

Table 3

The age of soil cement, τ	The module of deformation according to tangent x 10^{-2} in *MPa* under stress σ (*MPa*)			
	0	1.5	3	4.5
28 days	41.1	36.6	32.3	28.3
90 days	58.0	47.0	37.2	28.6

The comparison of the data introduced in Table 3 indicates that the observable great difference in Ē values of soil cement made at the age of τ = 90 days, τ = 28 days under σ = 0 (more than 41%) with the increase of

level observed in the compression stress σ gradually decreases while in the case of $\sigma = 4.5$ *MPa* the given difference practically disappears.

Conclusion

1. The relative difference of humidity between the test specimens of soil cement based on the mixture of white soil, carbonate and specimens of clay soil (for the production of which the cement of the same quality, exhibiting the same activity was used) has little impact on the growing durability of these specimens, the maturity process of which took place in the environment with low humidity (W≤75% [4]). This testifies to the fact that while making the mixtures of these types of soil cement much more water is used compared to the amount of water necessary for prompting chemical reactions in the production of cement stone. A similar practice is common for making mixture of cement concrete in order to provide concrete workability while moulding products in it.
2. The similar intensity of growing durability over time (starting from the aforementioned initial age τ) observed in the soil cement on the basis of white soil and carbonate mixture, in soil cement based on clay soil along with cement concrete where the same type of cement was used enables us to conclude that the given process (the process of gaining durability) mainly depends on the characteristics of the connective component in the cement used in the mixtures of these construction materials.
3. The process of ageing in the environment with high levels of humidity (in wet sawdust) has a favourable impact on the growing durability of the soil cement based on the mixture of white soil carbonate gain over time

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STUDY ON THE NEW METHOD OF CONSTRUCTING SHEAR WALLS IN MULTI-STOREY BUILDINGS WITH SITE CAST REINFORCED CONCRETE FRAME SYSTEM

The article touches upon the comparative analysis of bearing system calculations of a multi-storey residential building with site cast reinforced concrete frame and shear wall constructed by two different methods. In the calculation models, the shear walls are constructed from site cast reinforced concrete in the first case, and from three-layer sound and thermal insulating bearing panels in the second. The calculations have been made considering the impact of the seismic force. According to the calculation results, the dynamic parameters of the bearing systems of the buildings and the economic efficiency indicators have been compared. Considering the fact that in the recent years three-layer sound and thermal insulating panels have been widely used in the world, the study attempted to reveal the efficiency of using such panels in the Republic of Armenia.

Keywords: *site cast reinforced concrete, three-layer panel, shear walls, seismic impact, multi-storey building.*

Introduction

Along with the development of the construction industry, technologies of the implementation of building structures are developing, new building materials are emerging and studies are being conducted to develop more accurate schemes for calculating those new technologies and materials in the world. The integrity of these processes is aimed at improving the reliability of buildings, as well as their indicators of economic profitability.

The article examines the efficiency of using three-layer sound and thermal insulating panels, widely used in the world, as shear walls in multi-storey buildings constructed in the Republic of Armenia.

It should be noted that the three-layer panels are most often used as external covering structures in Armenia. At the same time, over the years, numerous studies have been carried out in the direction of technologies of calculation and implementation of this type of panels, special methods for the implementation and modeling of their nodes have been developed. Taking into account the aforementioned, we have performed numerical analyses of the bearing systems of buildings with or without three-layer panels in case of seismic impacts on the example of multi-apartment buildings. The nodes of the structures were modeled to meet the requirements of seismic construction codes and rules for designing reinforced concrete structures[1] [1,2]. Calculation models of buildings were developed using computer software operating on the basis of finite elements, and loads affecting the models were taken as their actual values[2] [3-8].

The three-layer panels under study consist of two outer 6 *cm* thick concrete layers, one inner polyurethane foam layer, in which the reinforcing frame is woven from longitudinal and transverse reinforcement bars with a special technology. The technology of manufacturing and installing panels is implemented in stages: at the first stage, reinforcing frame is woven through a special production line, at the second stage it is covered with

[1] HHSHN II-6.02-2006. Seysmakayun shinararut'yun. Nakhagtsman normer. Yerevan, 2006, p.67 (in Armenian).
[2] SNiP 2.01.07-85*. Nagruzki i vozdejstviya, FGUP CPP, Moscow, 2005, p.44 (in Russian).

polyurethane foam, at the third stage it is moved to the construction site and installed in the design position. At the fourth stage the panel is connected to the bearing frame by specially developed nodes and at the fifth stage it is covered with thick layers of concrete from both outer sides[3].

The appearance of the three-layer sound and thermal insulating panels used in the multi-storey building under study is presented below (Figures 1, 2).

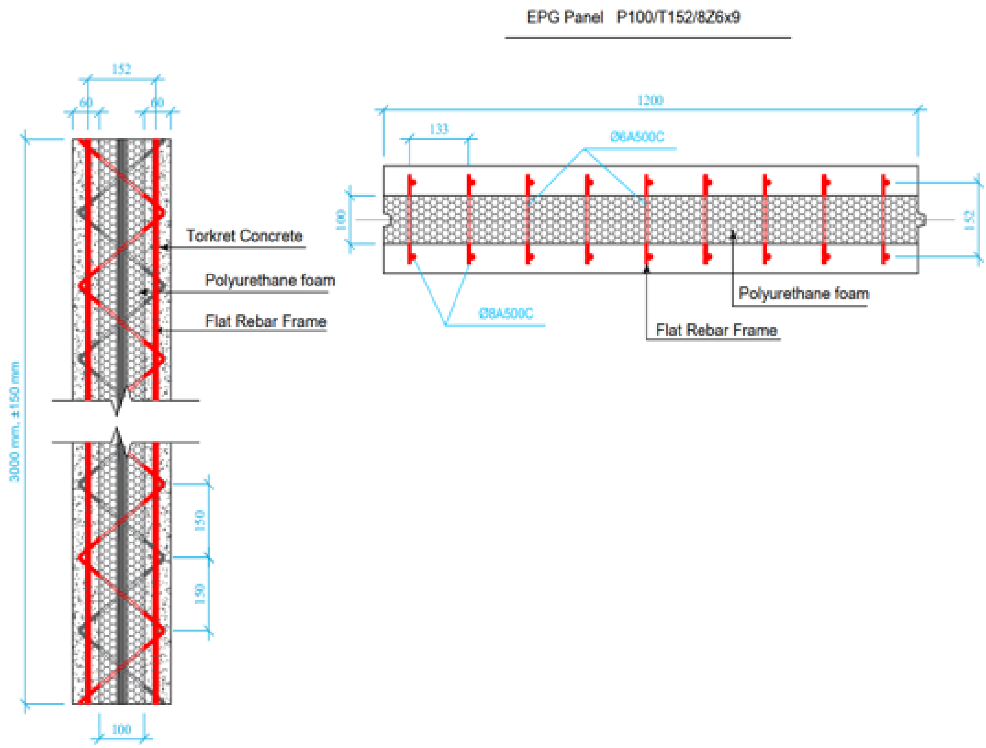

Fig. 1. *Transverse and longitudinal sections of three-layer sound - and thermal insulating panels*

Fig. 2. *Three-layer sound- and thermal insulation panels during installation*

[3] EPG Wall, Floor and Roof System by Eco Panel Group, ICC Evaluation Service, Report, Glendale, 2018, p.274.

Research part

The studies are carried out on the basis of calculation results of the bearing system of a single building of multi-storey residential complex with reinforced concrete frame, which is currently under construction in Yerevan[4] [1-8]. The bearing system of the multi-storey building under study has been modeled with two different shear walls – site cast reinforced concrete and three-layer bearing panels [1-8]. The article presents a comparative analysis of the results of the calculations made with the two aforementioned options. When considering the operation of the bearing system of a multi-storey building, the baseline data used for the calculations, such as the properties of the soil used as a foundation, seismic characteristics of the site, and the loads affecting the structure, were assumed to be the same. The calculations were made using a calculation software operating with the finite element method [7-10]. The appearance of the three-dimensional models of the residential complex and of one building under study is presented below (Fig. 3).

Fig. 3. *Three-dimensional models of the multi-storey residential complex and the building under study*

The following numerical studies were performed during the calculations:

- static calculation according to the first group limit state,
- static calculation according to the second group limit state,
- seismic calculation according to the first group limit state, taking into account the coefficient of allowable damages,
- seismic calculation to check the deviations.

Making the calculations, it was taken into account that:

- in case of static calculation, the concrete elasticity modulus is accepted with a coefficient of 1.0,
- in case of seismic calculation, the concrete elasticity modulus is accepted with a coefficient of 0.75,
- in the case of seismic calculation for buildings with reinforced concrete frame systems, when the allowable damage coefficient is $k_1 = 1$, the maximum deviation value should not exceed (1/300)•H (where H is the floor height), in this case the elasticity modulus in the calculation is accepted with a coefficient of 0.75. The regulating coefficient of 0.8 for displacements by HHSHN II-6.02-2006 has not been applied,
- when making calculations, it was taken into account that the magnitude of the ground subgrade coefficient - c_1, varies during static and seismic calculations [11-13].

The bearing system elements of the first design model have the following sections:

- Foundation - T-beam 120x30x60x60 c*m* 40x60 *cm* (E_b = 2.3x10^5 *MPa*),
- Column - 40x40 *cm*, (E_b = 2.3x10^5 *MPa*),
- Beam - 40x60 *cm* (in the direction of *X*), 40x40 *cm* (in the direction of *Y*) (E_b = 2.3x10^5 *MPa*),
- Roofing slab - 20 *cm* (E_b = 2.3x10^5 *MPa*),
- Shear wall - 40 *cm site cast* (E_b = 2.3x10), 22 *cm* three-layered (E_b = 2.0x10^5 *MPa*),
- Heavy concrete B25,
- Reinforcement bar A500C, A240,
- a_{crc} = 0.3 and 0.4 *mm* from the condition of preservation of the reinforcement bar.

[4] SNiP 2.03.01-84*. Betonnye i zhelezobetonnye konstrukcii, Moscow, 1998, p.80 (in Russian).

The elements of the bearing system of the second design model have the following sections:

- Foundation - T-beam 120x30x60x60 *cm*, 40x60 *cm* (Eb = 2.3x105 MPa),
- Column - 40x40 *cm* (Eb = 2.3x105 MPa),
- Beam - 40x60 *cm* (in the direction of X), 40x40 *cm* (in the direction of Y) (Eb = 2.3x105 MPa),
- Roofing slab - 20 *cm* (Eb = 2.3x105 MPa),
- Shear walls - 40 *cm* site cast (Eb = 2.3x10), 22 *cm* three-layer (Eb = 2.0x105 MPa),
- Heavy concrete B25,
- Reinforcement bar A500C, A240,
- a_{crc} = 0.3 and 0.4 *mm* from the condition of the preservation of the reinforcement bar.

In Table 1 the values of calculation loads are presented.

Table 1. Loads included in the calculations and their values

Load name	*Normative load, n/m^2*	Load reliability coefficient, γ_f	Calculated load, n/m^2
Permanent			
r/c slab 0.2 *m*	3000	1.1	3300
floor layers and partitions	3200	1.1	3500
Additional permanent load from the weight of the longitudinal partition of the building - 12002 *n/m*			
Temporary			
short term	2400	1.2	2880
long term	1200	1.2	1440

A three-dimensional image of the models developed by the LIRA software and the results obtained are presented below (Figures 4-8).

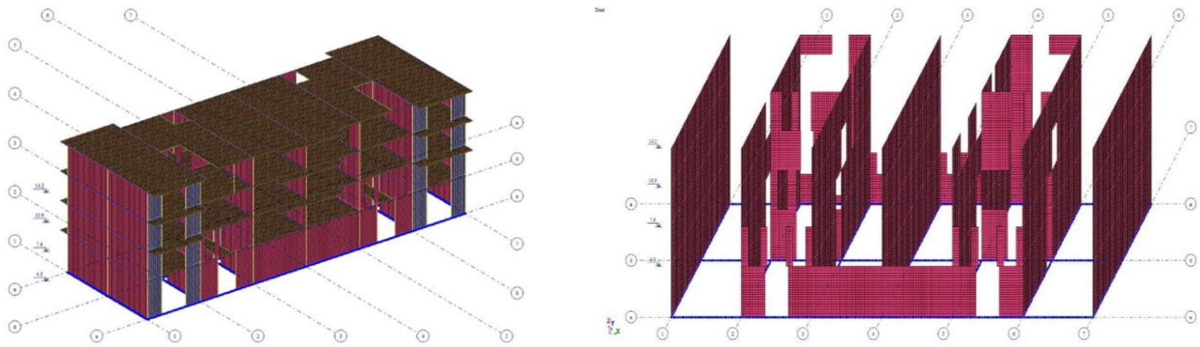

Fig. 4. *Multi-storey residential building model image developed by LIRA software (I, II models)*

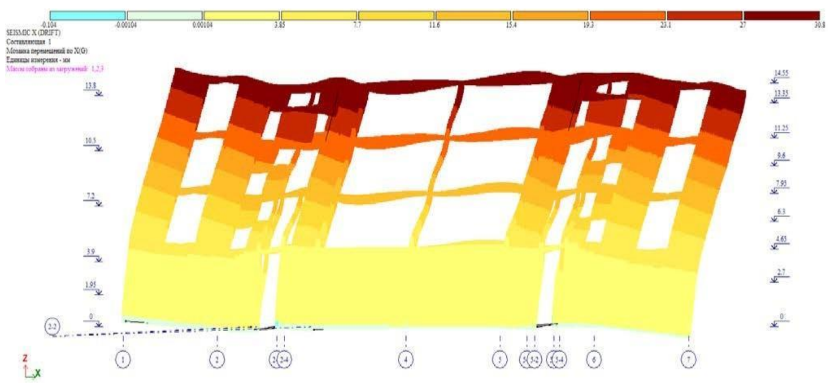

Fig. 5. *Analytical image of floor movements of the first model from seismic horizontal load in direction of X*

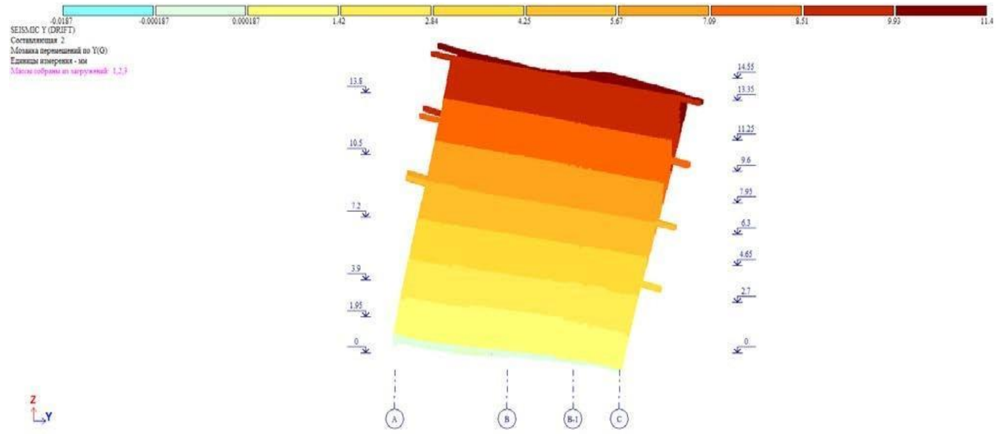

Fig. 6. *Analytical image of floor movements from seismic horizontal load in the direction of Y*

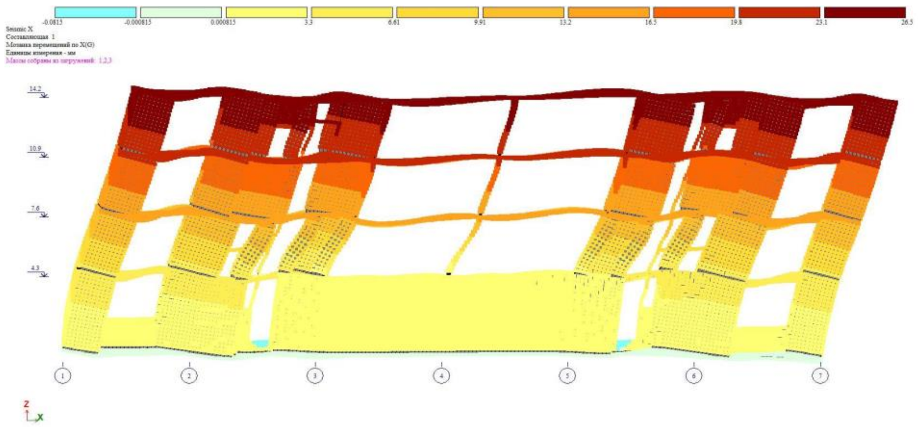

Fig. 7. *Analytical image of floor movements of the second model from seismic horizontal load in the direction of X*

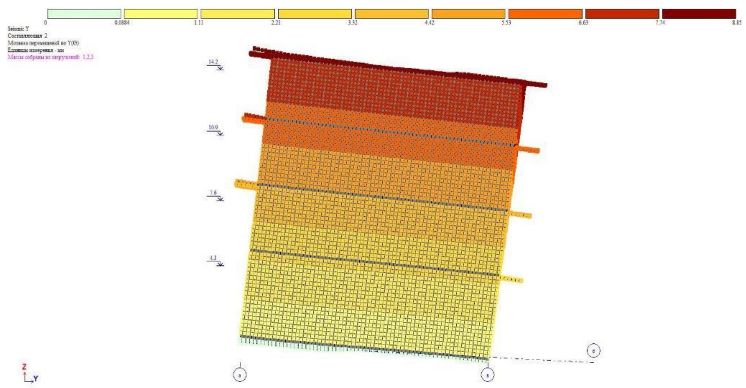

Fig. 8. *Analytical image of floor movements of the second model from seismic horizontal load in the direction of Y*

Results and Discussion

According to the analysis of the calculations, the strength and reliability of the proposed structure can be provided both with site cast reinforced concrete and in the case of shear wall modeling with three-layer panels.

Some of the data obtained from calculation results are presented below.

Table 2. Dynamic characteristics obtained by calculation for the structure

N	Periods of vibrations	Maximum floor displacement in the direction of X, *mm*	Maximum floor displacement in the direction of Y, *mm*
First model	0.302	8.3	3.37

Second model	0.267	7.2	2.09

Table 3. Shear wall material consumption according to calculation results

N	The total value of shear walls reinforcements, *t*	The volume of concrete used in shear walls, m^3	The volume of polyurethane foam used in shear walls, m^3
First model	13.581	137.28	144
Second model	21.425	228.8	0

As we can see it from Table 2, the second structural model is more rigid and has better indicators from the point of view of dynamic characteristics. According to Table 2, the volume of reinforcement bar required in the first model is about 25% smaller than in the second model. In terms of concrete consumption, the first model saves 90 m^3, but increases the cost of 114 m^3 of polyurethane foam. By reducing the volume of concrete, the natural weight of the first model is reduced by 220 tons.

Volumes of the reinforcements of other elements of bearing systems of design models are not given in Table 3, since their differences do not exceed 1.5% and do not make any sense in terms of comparison.

Conclusion

According to the obtained data, during the comparative analysis it becomes clear that the bearing system of the structure with site cast reinforced concrete shear walls is more rigid than the bearing system of the structure with three-layer panels, therefore the first model has small floor deviations. It can be concluded that under favorable soil conditions it is more expedient to make shear walls from three-layer panels, taking into account the profitability of their economic indicators.

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THE IMPORTANCE OF HISTORICAL STAGES OF CONSTRUCTION WITHIN THE CONTEXT OF SUSTAINABLE URBAN DEVELOPMENT OF HADRUT CITY

Preservation of Artsakh's architectural heritage and the issues of use have acquired a special significance today. The historical stages of the construction of the city of Hadrut in Artsakh, and, as a result of their analysis, their impact on the further sustainable development of the city have been revealed. There are 5 main stages: formation, late Middle Ages, from the 19th century to the first half of the 20th century, Soviet and independence periods. Suggestions are given on the territorial development of the city.

Keywords: *urban development stages, historical stages of construction, sustainable development, architecture of residential buildings, Hadrut city, cities of Artsakh.*

Introduction

The historical development of the city is the most important part in the context of the creation and sustainable development of the city. Many cities in Artsakh do not have a development strategy or conception, which can lead to the wrong choice of area for modern construction. The study of the existing historical settlements and the problems of their construction is of great importance for solving the problems of the territorial development of Artsakh. During the Soviet period, Azerbaijan pursued a policy of concealing and destroying the Armenian historical and cultural heritage in Artsakh. This approach has had a negative impact on the spatial development of the cities. In recent decades, the relatively slow urban development of Artsakh has contributed to the preservation of the architectural heritage of historic Armenian cities, as well as to the holistic historical and cultural architectural image of the region up to the present day. The problem of preserving and using this valuable heritage has acquired special significance today.

The problem of preserving and using the historical construction of Hadrut is hindered by its lack of knowledge. It should be noted that unique complete samples of the Armenian national habitat have been preserved in Hadrut, which are a vivid example of the process of historical change of habitat. The study of dwellings provides an opportunity to study the types of social life, the formation of rural communities, their origins, traditional rules of behaviors and communication skills. During the study of the habitats, the fact of the fragmentation of their types and the ways of the process of transformation are of interest. There are examples of housing construction and habitats not far from each other, the principles of development and substantiation of which are not easy to understand. Moreover, the possible architectural relationships with neighboring countries, such as Iran and Russia, the influence of which within the normal range lasts for decades, have not been studied.

Before the 44-day war, a new wave of construction had begun in Hadrut city of Artsakh, but there was no plan to preserve the city's identity. The modernization process did not take into account the historical and cultural heritage and the architectural plan of the city. This led to a number of urban-architectural planning problems. Although I. Davitbekov wrote about the city of Hadrut in his work “The village Hadrut” in 1887, nevertheless there is no work revealing the historical development of the city and its overalls.

In the 1980s, Manvel Sargsyan implemented measurements of settlements with his own financial resources. Arthur Mkrtchyan's contribution to the collection of information on the foundation and development of the city is also invaluable. However, in Hadrut, the structures of historical and architectural value are being destroyed: in particular, houses built at the end of the 19th century, which are not protected by the local self-government bodies, have been forgotten.

Materials and Methods

Materials and methods of the research are based on the study of archival, historical and literary materials and on the author's situational observations and analysis.

Results and Discussion

In order to understand the stages of territorial development of Hadrut city, it is necessary to analyze the stages of city formation, historical development, external influences and other factors.

The period from the approximate formation of Hadrut to the present day can be divided into five stages, which include generalized historical events and the process of architectural planning development intertwined with the latter, as well as the constructions specific to each period.

First References and Formation. 14the-16th centuries

The earliest reference on Hadrut known to us dates back to 1428. In that year, in the "Chgnavori qar" desert near the historical city of Yeghegis, the center of the historical Vayots Dzor, a gospel manuscript was copied. The gospel was penned by Yeghia, who, as it could be seen from the memoir, was originally from Hadrut[1].

The manuscript was written "ի թվին Հայոց ՊՀԷ" [1] which corresponds, as it has already been mentioned, to the year of 1428, that is, about 560 years ago. However, if we take into account that Father Yeghia wrote the manuscript in mature age, and he is from Hadrut only by origin, it would mean that the fact that Hadrut is not less than 600 years old is a historically documented fact. However, it is natural that the first reference does not mean that it immediately followed the establishment of the settlement. Newer or older references may be found in those unknown sources. There is evidence that during the construction the remains of the pagan period and khachkars (cross-stones) were found in Hadrut[2]. Unfortunately, they have not been studied at all.

According to the aforementioned information by Arthur Mkrtchyan, one can claim that the village of Hadrut was formed before the 15th century. However, very few architectural structures, mainly the ruins of castles and churches, have been preserved from this period. It is noteworthy that the fragments of dozens of monuments found during field observations refer to both pagan and early Christianity periods as well as the Middle Ages. According to the description of Hadrut in Sh. Mkrtchyan's book "Historical and Architectural Monuments of Nagorno Karabakh", five main buildings that had been typical to Hadrut are highlighted, Hin Hangstaran, Ghalin Band, Tschakhach Ghalay, Vnesa Ghalay, Spitak Khach Monastery[3].

The Hin Hangstaran, is now partially preserved and is located in the western part of the village. Tombstones without records indicate their pagan origin. According to Sh. Mkrtchyan, only two tombstones were found, on which a cross was engraved. The latter also mentioned that in the 19th century these tombs were called "Krapashti hangstaran"(Idolater's cemetery) among the people.

In the northeastern part of the village is the Ghalin Band (entrance to fortress) castle with towers, which is not preserved now.

About 1.5 *km* away from the previous one, on the slope of a small hill, there are the ruins of another fortress called "Tstsakhach Ghalay". Once there was a great need for water in this fortress. This is evidenced by the fact that water from the village of Shaghakh (Saren-Shen) was brought artificially by the inhabitants of that time, as evidenced by the remnants of clay pipes, which here are called "Tyungi". The tomb of the owner of this castle Velijan, is located near the Church of White Cross. The tombstone was laid by Velijan's son Khumar in 1527 [2].

To the northwest of the village, on a high forested "Vnesa-mountain", there is another ancient fortress called "Vnesa Ghalay". This means that it belonged to the owner of that time Vanes (Hovhaness). Only the half-ruined church and the walls of the former building remained now.

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[1] A.A. Mkrtchyan, Arrajin hishatakut'yunnery Hadrut'i masin (unpublished manuscript). Hadrut, 1987, p.150 (in Armenian).

[2] Ibid.

[3] Arts'akhi k'artezagirk'. http://www.raa-am.com/raa/pdf_files/169.pdf

Spitak Khach Monastery is located a few hundred meters south of Hadrut, near a village called Vank, which is now part of the city of Hadrut. The village has been given that name probably because it is close to the monastery. And the monastery complex has existed since the 14th century [3]. The exact date of the construction of the church is unknown. However, it should be assumed that it was built in the 14th century at the latest, since it contains many traces of the 14th century. For example, the inscription of the cross under the north arch inside the temple: "Ես Սիրաւք կանգնեցի զխաչս հաւր իմոյ Խութլափի թվ. ՉԶԲ (1333)" [4].

These 5 architectural structures date back to the same period, 13th-16th centuries, they surround Hadrut, but only the Spitak Khach Monastery has been preserved. Another reference on Hadrut is found in another manuscript of the Gospel written about 150 years ago from the previous one, which is now kept in the manuscript repository of the Zmmar Monastery near Jerusalem. This manuscript was already copied in Hadrut by Hieromonk Abraham in 1584[4].

The manuscript already proves that in the 16th century Hadrut was a famous settlement, the center of writing, that is, there was some famous building here[5], probably it was the church in the old cemetery, which can be supposed to have been a writing center.

This conclusion is confirmed by the fact that other manuscripts written in the villages of Tyak-Taghaser are known from the 16th -17th centuries. It is natural that these villages, being located very close to each other, were in close contact[6].

Based on this information, the outline of Hadrut of that period is presented, including a number of constructions that were described in the work "The village Hadrut" written by I. Davitbekov in 1887 (Fig. 1).

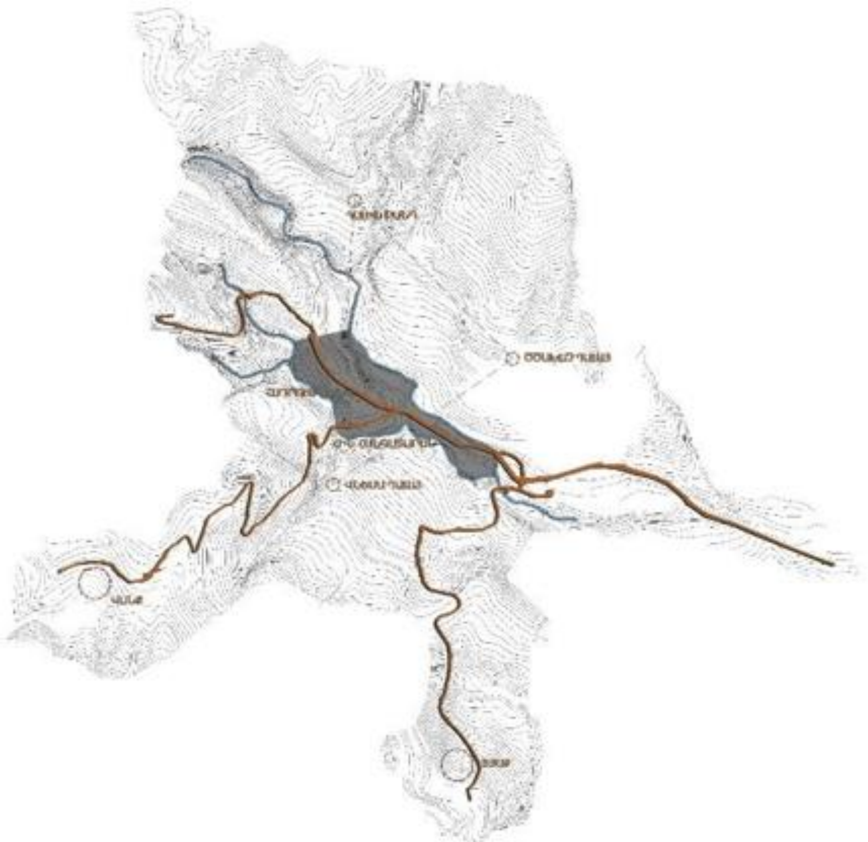

Fig. 1. *Hadrut outline in the 16th century according to historical sources and location of castles*

Late Middle Ages. 17th-18th centuries

There is no complete information about the architecture of Hadrut in the 17th-18th centuries, but various famous people visited Hadrut and referred to the fact that the village was developed compared to the

[4] A.A. Mkrtchyan, Hadrut'i ants'yalits' (unpublished manuscript). Hadrut, 1987, p.96 (in Armenian)..

[5] Ibid.

[6] Ibid.

surrounding villages. The aforementioned information indicates that already in the 18th century Hadrut was a well-known settlement and was famous outside Artsakh.

St. Harutyun Church is located in the historically formed part of Hadrut – in the circle of old buildings. According to some sources, it is a large and durable structure, the roof of which is covered with polished semicircular stone slabs.

This church was built in 1621. Polished stones and cross-stones belonging to the ancient monastery, which are typical to the 12th-14th centuries are used inside the walls [2].

Ancient Hadrut was surrounded by a number of defensive fortress-castles. In the north-east, the traces of the old fortress named "Berdi mutq" have been preserved, separate parts of walls and towers of "Tstsakhach" fortress in the south, "Vnesaberd" in the north, "Tsoraberd" in the southeast have also been preserved. Hadrut, along with the province, has been repeatedly destroyed by Persians, Arabs, Seljuk Turks, Tatar-Mongols and Ottoman Turks, but has always been rebuilt. This is evidenced by the lithographs of Spitakakhach Monastery, Shakhkakh, Shinategh, Anapat, St. Harutyun, St. Astvatsatsin, Grigor the Illuminator, Amenaprkich, St. Grigor monasteries and churches (9th-17th centuries) and many other monuments in Hadrut and the surrounding villages [2].

The conducted researches allow to restore the outline of Hadrut in the 17th-18th centuries (Fig. 2).

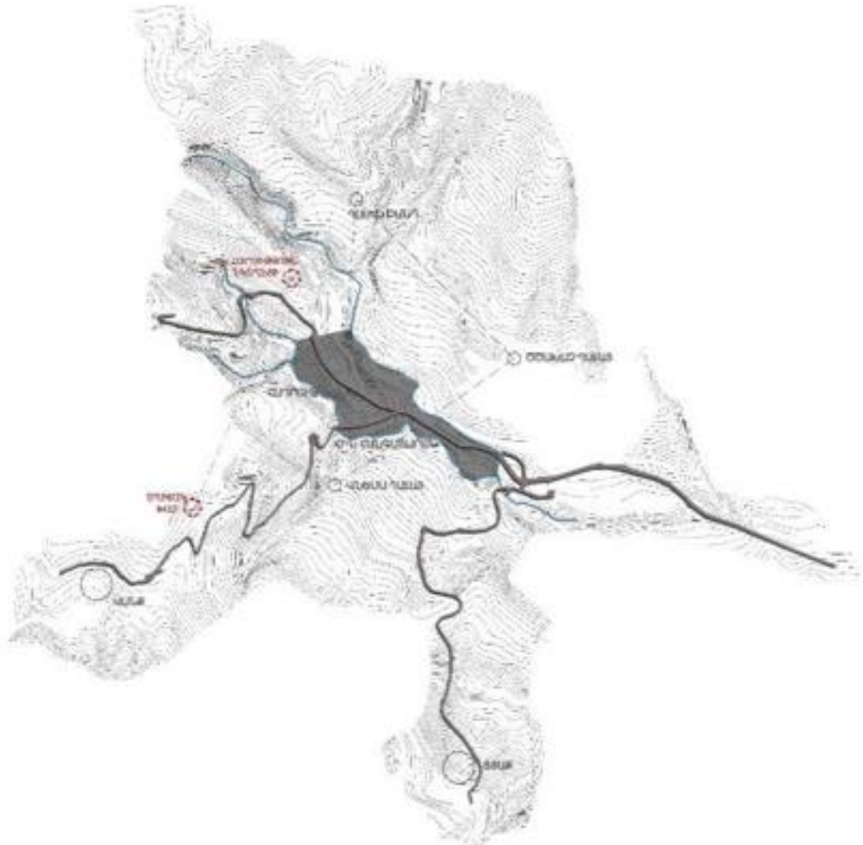

Fig. 2. *Hadrut outline in 17th-18th centuries*

The period from the 19th century to the first half of the 20th century

The 19th century was especially conducive to the prosperity of Hadrut, after the unification of Eastern Armenia with Russia, when the population of the barracks quadrupled over several decades.

In the past, Hadrut was not so different from surrounding villages, but since the establishment of the military unit, some residents were able to earn much money by delivering various supplies for the headquarters.

Such residents began to build one-storey and two-storey houses with tin roofs to the best of their abilities. But the houses of the ordinary people were built under a conical roof leaning on wooden pillars. The pillars were arranged very close to each other, so that from a distance it seemed to be a common wall, and the conical openings for light and smoke penetration were also visible. The surrounding forest, stones, sand and clay, as well as a small amount of fired brick were a source of construction materials. The forest is located 10-15 *km* west of Hadrut, and the other materials were obtained directly from Hadrut [5,6].

On the whole, there was only one street in the village, that started from the headquarters (military unit) and before reaching the middle of the village, it was narrowed, not allowing to pass with a carriage. The remaining streets were narrow alleys paved with tiles.

Thus, although Hadrut was not a provincial center, it played an important administrative role in the life of the province. This circumstance made a great contribution to the development of the settlement. And it is not accidental that in 1880 a telegraph office was opened in Hadrut, and in 1885 a pharmacy and a four-bed patient reception center, which served the entire province of Jabrayil.

The cultural life of the settlement is marked by the fact that in 1879 an Armenian parish school was opened in Hadrut, and two years later also a Russian two-class school.

Later, in 1912, due to the efforts of the teacher of the Armenian school in Hadrut S. Nazaryan, a branch of the Armenian Benevolent Society of Caucasus was created, one of the first actions of which was the opening of a library-reading hall. The economic life of Hadrut was becoming more active. A silk factory was established in 1877 (Fig. 3).

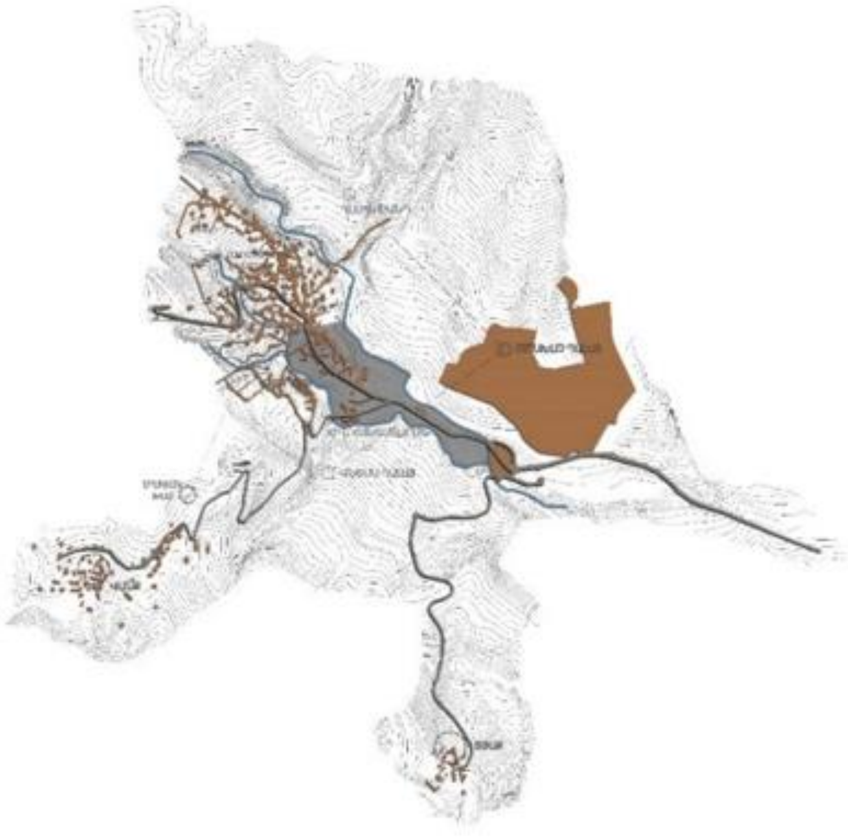

Fig. 3. *Hadrut outline according to the construction carried out from the 19th century to the first half of the 20th century*

The settlement was gradually becoming bigger and the population was growing. If in 1823 the population in Hadrut was only 229 people, then at the beginning of the 20th century it had increased tenfold reaching 2,300. Along with the administrative, economic and cultural development and growth of Hadrut, the settlement and life of the inhabitants were improving. In 1819, the St. Harutyun church of the village was

renovated - a dome was installed on it[7]. In 1900 a spring was built, in 1908 a two-span bridge was built on the river passing by the settlement near Khor Aghbyur, the central street was tiled, buildings of Armenian and Russian schools and administrative buildings were constructed. Residents began to build one-storey and two-storey town-like houses, furnishing them with European furniture.

The available materials allow us to claim that at the beginning of the 19th and 20th centuries, Hadrut was one of the most famous settlements in Artsakh after the city of Shushi (Fig. 3). In the 19th century, Hadrut was constructed on a regular planned basis. The old part of the settlement is still visible today. From the lower part of the regional center to the upper cemetery, on both sides of the street, valuable one- or two-storey balcony houses and shops of folk architecture are preserved. According to the formed list, their number reaches about 300 [2].

Soviet period. 20th century

In 1918, the people of Hadrut showed strong resistance to Turkish forces attacking Zangezur. After the Sovietization, Hadrut was forcibly incorporated into the Azerbaijani SSR, and deprived of the possibility of its development. Almost all industrial enterprises were closed, hundreds of hectares of fertile mulberry orchards were cut down or uprooted, and a number of buildings were demolished.

Later, a number of buildings were built in Soviet times. These include the House of Culture, built in 1980, the Cinema in 1971, the Winery in 1960, the Editorial building in 1950 and the Museum building in 1930 (Fig. 4).

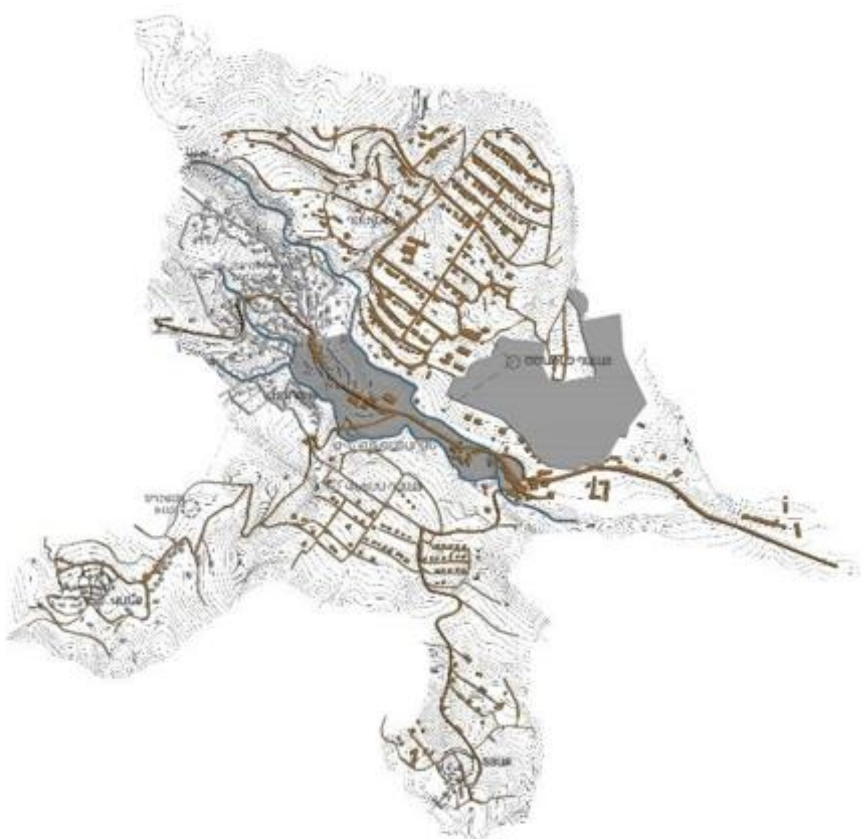

Fig. 4. *Hadrut Outline during the Soviet era*

Independence period. 1990 – today

Thanks to the Karabakh movement of 1988, the population of Hadrut experienced a national revival. Residents of the city fought together with the Armenians of Artsakh for their freedom. Indeed, over the past

[7] List of monuments. http://www.monuments.nkr.am/

30 years, some buildings that do not fit into the urban landscape have been constructed, but the city has entered a new stage of development (Fig. 5).

Due to the 2020 war, the city of Hadrut was handed over to the Republic of Azerbaijan, as a result of which the Artsakh Republic lost its territorial integrity, as well as the possibility for the further study of the architecture for an indefinite period of time.

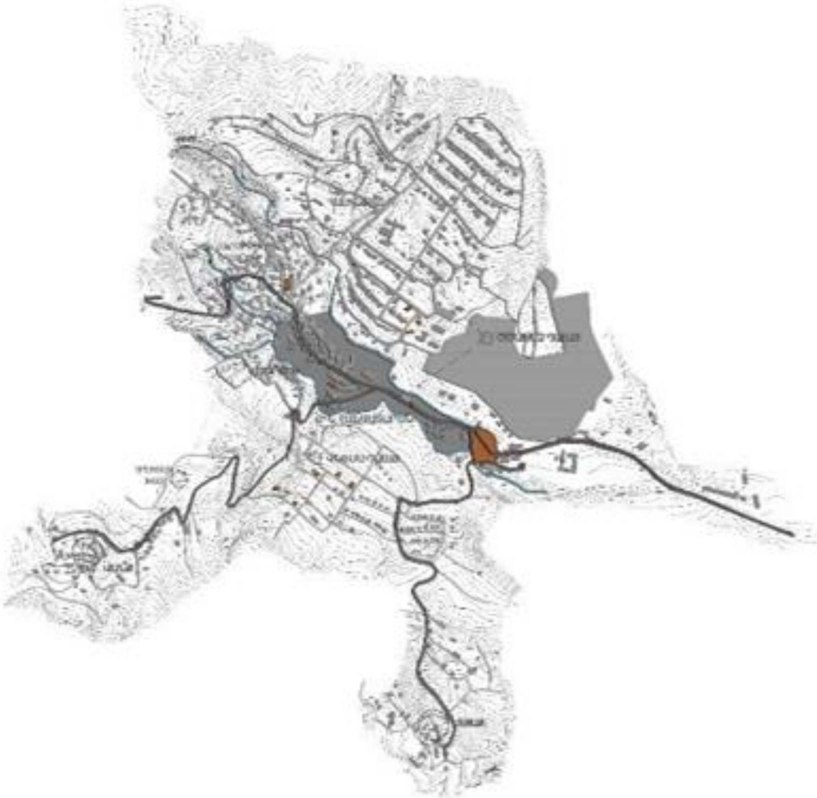

Fig. 5. *Hadrut Outline after independence*

Conclusion

As a result of the research and analysis, five main stages of urban development are distinguished together with the existing architectural monuments belonging to each of them:

- First references and formation: 14th-16th centuries
- Late Middle Ages: 17th-18th centuries
- The period from the 19th century to the first half of the 20th century
- Soviet period. 20th century
- Independence period. 1990 - today

The architecture of this ancient settlement, which has a great historical value has not received due attention, and currently systematic research is hampered by the destruction caused by the war and the occupation of the territory by Azerbaijan.

However, in case of solving geopolitical problems, taking into account the above-mentioned circumstances, all further construction works should be relocated to another area, in order not to damage and interrupt the panorama of the old city by modern construction (Fig. 6).

It is proposed to expand the city to the south-east and north-west, where it is possible to develop the city without damaging the panorama of the old district. Analysis of the historical stages of Hadrut construction makes it possible to make a correct and justified choice of territories for further development of Artsakh cities through such methods and study [7].

Fig. 6. *Proposal for further development of Hadrut outline*

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STIFFNESS OF POST-TENSIONED GIRDERLESS FLOORS WITH DIFFERENT COLUMN GRIDS

The paper considers models of monolithic flat floor slabs with five spans in both directions. The cell sizes are 6×6m, 6×9m, and 6×12m. The calculation method is based on the application of temperature load and rope modeling of rod elements. It is shown that post-stressing should be used for slab side lengths over 7 m, as the installation of pre-stressed reinforcement for shorter lengths is less feasible and causes high economic costs.

Keywords: *reinforced concrete slab, stiffness, deflection, column grid, post-tension.*

Introduction

When designing structures with a prestressed system without adhesion to concrete, it should be taken into account that prestressed reinforcement does not transfer forces to the concrete over the entire length, but only at the anchor points at the ends of the structure, as well as at the bends in the ropes. Accordingly, the prestress must be assumed in the calculation as external forces applied to the structure. The forces formed at rope bends depend primarily on the rope geometry and the forces in it.

Works [1-8] are devoted to studies of girderless structures with prestressed reinforcement.

The calculation of prestressed elements in deformations (stiffness) is carried out according to the normative document[1]. The compression force *Np* is determined by taking into account all losses and $\gamma sp = 0.9$. Deflections are calculated by considering the strength of concrete at different stages of loading, including the transfer of compression forces.

The deflections of reinforced concrete elements are calculated under the condition:

$$f \leq f_{ult}, \tag{1}$$

where f is the deflection of the reinforced concrete element from the external load,

f_{ult} is the value of the maximum permissible deflection of a reinforced concrete element.

For bendable elements of a constant cross-section, along the length of the element without cracks, the deflections are determined by the general rules of structural mechanics using the stiffness of the cross-sections determined by the formula:

$$D = E_{b1} \cdot I_{red}, \tag{2}$$

where E_{b1} is the deformation modulus of compressed concrete, determined according to the load duration and taking into account the presence or absence of cracks,

I_{red} is the moment of inertia of the given cross-section in relation to its center of gravity, determined taking into account the presence or absence of cracks.

Materials and methods

The calculation method is based on the temperature load application and the modeling of ropes with rod elements [9].

[1] Posobiye k SP 63.13330.2012 «Konstruktsii zhelezobetonnyye monolitnyye s napryagayemoy armaturoy bez stsepleniya s betonom. Pravila proyektirovaniya» (in Russian).

The modeling of the reinforcement of the central cell structure is carried out using a rod element in LIRA SAPR. The cross-section of the rod element is similar to the area of prestressed reinforcement in the floor slab as well as to the computer synthesis [10].

In this paper, models of flat floor slabs with five spans in both directions are investigated. The cell sizes are 6×6*m*, 6×9*m* and 6×12*m*, with the thickness of h = 0.3*m* (Fig.1). The size of the finite elements is 0.3×0.3*m*.

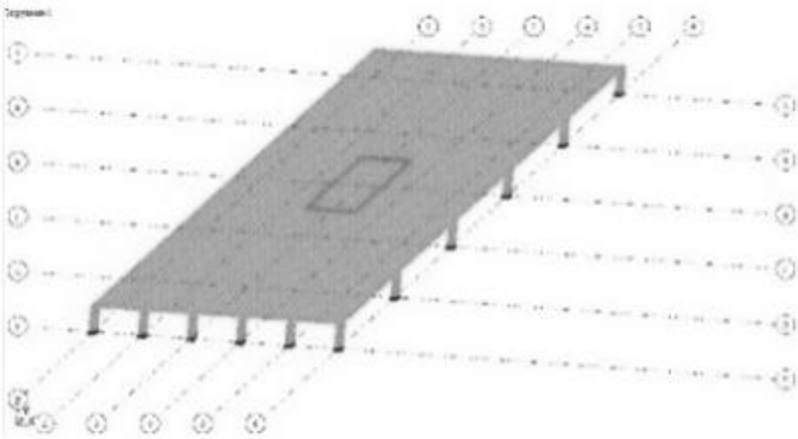

Fig. 1*. Scheme of a monolithic floor with rope-mounted reinforcement of the central cell*

Main characteristics of construction materials

Concrete class B30. Post-tensioned reinforcement in form of ropes (monostrends): K7O ($R_{sp,n}$ = 1860·10^3 kN/m^2, E_{sp} = 1.95·10^5 MPa, d = 15.7*mm*, Asp = 1.54cm^2). For calculation 5, 7 and 9 ropes are taken.

A uniformly distributed load q = 5 kN/m^2 is applied to the slab.

The reinforcement of the central cell is modeled by the rod elements in the structure, the section of which is similar to the prestressed reinforcement area in the element. To simulate prestressing in the reinforcement, a temperature load is applied to it, which is calculated according to the formula:

$$\Delta t = \varepsilon_0 / \alpha , \tag{3}$$

where $\varepsilon_0 = \sigma_0 / E_p$,

E_p is the modulus of elasticity of the prestressed reinforcement,

σ_0 is the controlled tension of the prestressed reinforcement,

α is the expansion coefficient of the reinforcing steel.

The deflections will be considered at characteristic points of the cell (Fig. 2).

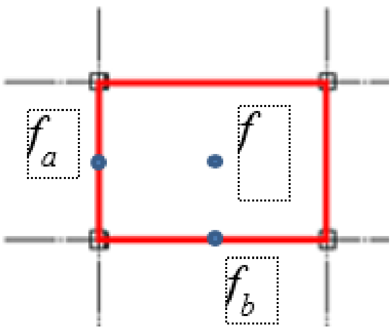

Fig.2. *Characteristic points of the central cell: f - deflection in the center of the cell, f_a - deflection in the center of the smaller side of the cell, f_b - deflection in the center of the larger side of the cell*

The prestress, calculated by the formula (3), is equal:

$$\varepsilon_0 = \frac{0.7 \cdot 1860}{1.95 \cdot 10^5} = 667 \cdot 10^{-5}.$$

$$\Delta t = \frac{667 \cdot 10^{-5}}{0.000012} = 556\ {}^0C.$$

The reinforcement of the structure's central cell was modeled using a rod element in LIRA SAPR 9 (Fig. 3).

A 6x6*m* slab cell with 5 ropes along the contour is modeled using flat (floor slab) and volumetric (columns) elements. The size of the finite element is 0.3x0.3*m*.

The floor slab is defined as a plate with a thickness of 300*mm*. The section of the column is 600x600*mm*.

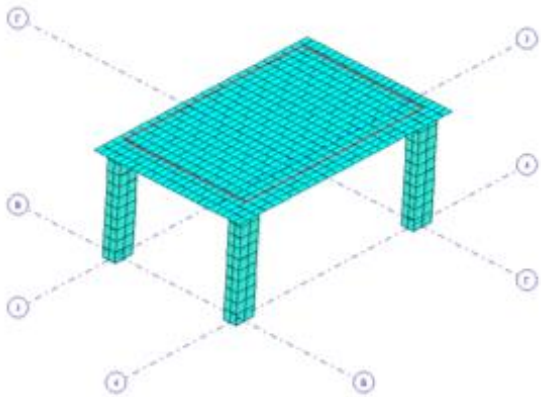

Fig. 3. *Calculation scheme for a 6x6m central cell with 5 ropes*

The results of the calculation are shown in Figures 4 and 5.

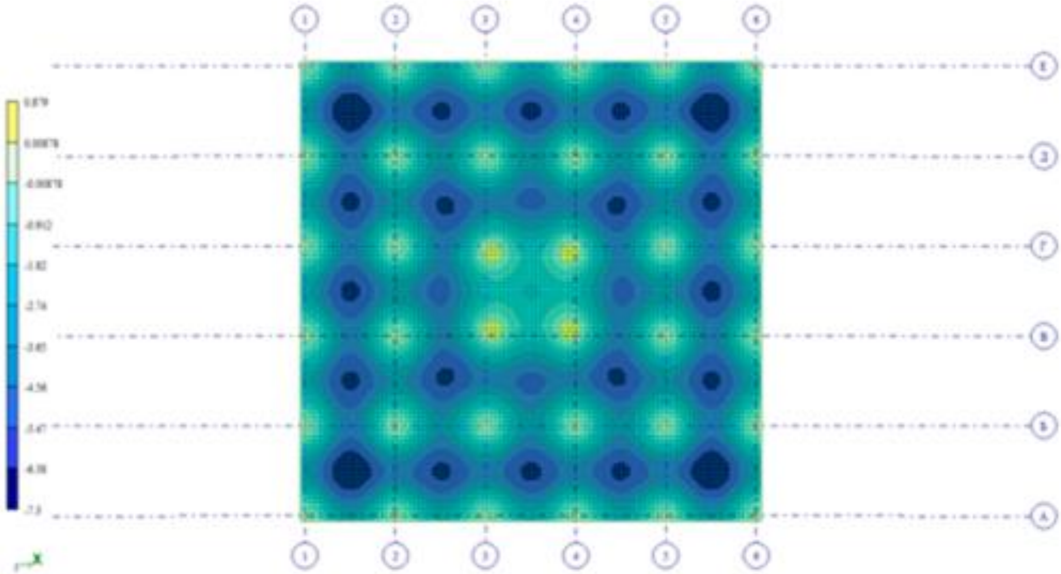

Fig. 4. *Travel along the Z axis of the entire slab with 6x6m cells and a central cell reinforced with 5 ropes*

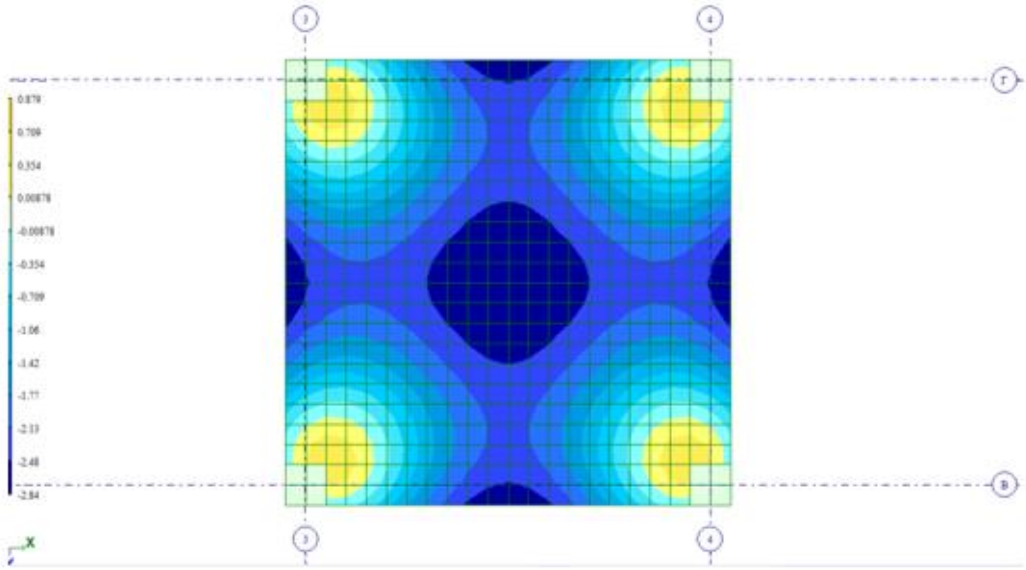

Fig. 5. *Travel along the Z axis of the central cell reinforced with 5 ropes*

Results

Based on the calculations carried out for the central cells 6x6, 6x9 and 6x12*m* with different numbers (5, 7, 9) of ropes, the deflection values were obtained, which are summarized in Table 1.

Table. Deflections of central cells with different numbers of ropes

Cell, *m*	Deflections (5 ropes), *mm*			Deflections (7 ropes), *mm*			Deflections (9 ropes), *mm*		
a×b	f_1	f_{a1}	f_{b1}	f_2	f_{a2}	f_{b2}	f_3	f_{a3}	f_{b3}
6×6	2.84	2.52	2.52	1.81	1.88	1.88	0.936	1.43	1.43
6×9	14.4	5.03	14.8	13	3.99	13.7	11.7	3.52	12.7
6×12	44.9	5.7	46	42.1	4.95	43.6	39.6	4.29	41.5

Table 1 shows that for a 6×6*m* square cell, with an increase of prestressing ropes from 5 to 7, the deflection in the center decreases from 2.84 to 1.81*mm* (1.57 times or 36.3%), and for 5 to 9 ropes - from 2.84 to 0.936 (3.03 times or 67%), in the center of "a" and "b" sides with an increase of the ropes from 5 to 7, the deflection decreases from 2.52 to 1.88*mm* (1.34 times or 25.4%) and for 5 to 9 ropes - from 2.52 to 1.43 (1.75 times or 43%).

For a 6×9*m* rectangular cell, with an increase of prestressing ropes from 5 to 7, the deflection in the center decreases from 14.4 to 13*mm* (1.1 times or 9.7%), and for 5 to 9 ropes - from 14.4 to 11.7 (1.23 times or 18.8%), in the center of the short side "a" with an increase of the ropes from 5 to 7, the deflection decreases from 5.03 to 3.99 *mm* (1.26 times or 20.7%), and for 5 to 9 ropes - from 5.03 to 3.52 (1.43 times or 30%), with an increase of the ropes from 5 to 7 in the center of long side "b", the deflection decreases from 14.8 to 13.7*mm* (1.08 times or 7.4%) and for 5 to 9 ropes - from 14.8 to 12.7 (1.17 times or 14.2%).

For a 6×12*m* rectangular cell, with an increase of prestressing ropes from 5 to 7, the deflection in the center decreases from 44.9 to 42.1*mm* (1.07 times or 6.2%), and for 5 to 9 ropes - from 44.9 to 39.6 (1.13 times or 11.8%), in the center of the short side "a" with an increase of the ropes from 5 to 7, the deflection decreases from 5.7 to 4.95*mm* (1.15 times or 13.2%), and for 5 to 9 ropes - from 5.7 to 4.29 (1.33 times or 24.7%), in the center of long side "b" with an increase of the ropes from 5 to 7, the deflection decreases from 46 to 43.6*mm* (1.06 times or by 5.2%), and for 5 to 9 ropes - from 46 to 41.5 (1.11 times or by 9.8%).

The results show that with an increase of the long side of the slab cell from 5 to 7*m*, the effect on the deflection of the installation along the contour of a larger number of ropes is reduced significantly. For example, for the center of the cell, the effect on the deflection of installing 9 ropes compared to 5 ropes is reduced from 67% to 11.8%, for the short side "a" - from 43.3% to 24.7%, for the long side "b," - from 43.3% to 9.8%.

Conclusion

1. Prestressing is recommended to apply when the side of the slab is longer, than 7*m*, since with a shorter side length, the installation of prestressing reinforcement is less expedient and entails high economic costs.
2. The use of post-tensioned ropes on the shorter side of a rectangular cell is unreasonable since the deflection on the short side meets the normative value even before the prestressed reinforcement is introduced into the model.
3. It is expedient to install more ropes for longer cells. For example, according to the calculation, it is not reasonable to install less than 7 ropes for a 6×9*m* cell, and it is expedient to install 9 ropes or more for a 6×12*m* cell. However, the use of 9 or more ropes can create large flexures at the base of the column, which would require additional reinforcement and strength testing of the compressed concrete.
4. With an increase of the long side of the slab cell from 6 to 12*m*, the effect on the deflection of the installation along the contour of larger number of ropes decreases significantly: for the center of the cell, it reduced from 67% to 11.8%, for the short side "a" - from 43.3% to 24.7%, for the long side "b" - from 43.3% to 9.8%. This indicates that for cells with sides of 7*m* or more, it is more expedient to use a average number of ropes but with a higher prestressing force to reduce the financial cost of their installation.
5. The method of modeling prestressed ropes using rod reinforcement with the application of a temperature load is simple, accurate, and easy to use.

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THE BASICS OF DEVELOPING AN ALTERNATIVE CONCEPT FOR COMMERCIAL AND MILITARY VEHICLE OPERATION (RANDOM STRATEGY)

The efficiency of commercial automobiles and wheeled military vehicles mainly depends on the choice of maintenance (M) and current repair (CR) concept. In the paper the difficulties of adapting the (M) and (CR) planning strategies to the structural characteristics of modern transport facilities are pointed out. The advantages of using the (M) and (CR) random strategy for transport facilities based on the stochastic nature of failures and malfunctions are substantiated. Considering the failures and malfunctions as random values and identifying the patterns of their distribution based on γ percentage resources, it is proposed to develop a list of regulated maintenance and repair work, periodicity and labor intensity based on a random strategy, which will increase the efficiency of preserving the technical resource of the rolling stock throughout the entire life cycle of the vehicle.

Keywords: *random strategy, resource, regulated service, adaptation, probability of unfailing operation.*

Introduction

Current provisions on the technical operation of modern commercial automobiles and wheeled military vehicles, the underlying concept, the normative basis and technology do not correspond to the actual processes of preventing malfunctions and failures. The main reason for the situation is the discrepancy between the list of preventive maintenance, performance frequency, and labor intensity that correspond to the actual necessity. This is due to new constructive solutions for transport facilities and qualitative changes in the exploitation materials.

Materials and methods

The provisions in the current situation[1] and the "plan" for the operation of wheeled military vehicles[2], the fixed normative basis, the frequency of work, the list of works, and labor intensity often lead to unnecessary work, which increases labor and material costs and inefficient downtime, as well as reduces the vehicle readiness index. As a descriptive example, we can mention the labor intensity for the operation of automobile engines, braking systems, transmissions and other operating regulations, which is practically unnecessary, as they are solved by structural innovations (mechanisms for automatic adjustment of gaps, management of engine crankcase and transmission oil coolant levels, and electronic electromechanical control of temperature, etc.) [2,6]. The labor intensity of these works is 8 ÷ 12% in the maintenance list (M2), while the implementation frequency does not exceed the value of 0.1-0.2.

Due to lack of functionality, many auto parts of military transport facilities eventually lose their airtightness and physical and mechanical properties, especially rubber technical machine components, bushings, piston rings, valves of all types, springs, engine power system, etc., where a maintenance list for additional work and

[1] Polozheniye o tekhnicheskom obsluzhivanii i remonte podvizhnogo sostava avtomobil'nogo transporta (approved by the Ministry of Transport of the RSFSR), 1984 (in Russian).
[2] Voyenno-tekhnicheskoye informirovaniye. Plan-konspekt komandirskoy podgotovki po voyenno-tekhnicheskoy podgotovke. Organizatsiya ekspluatatsii, remonta i khraneniya avtomobil'noy tekhniki, 2019 (in Russian). https://shtab.su/konspekt/voenno-texnicheskoe-informirovanie/organizaciya-ekspluatacii-avtomobilnoj-texniki.html

labor-intensity is necessary to develop. It follows from the presented example that the concept of technical operation of wheeled military vehicles based on the precautionary strategy no longer ensures the efficient technical operation of both commercial and military transport facilities with a high level of reliability, in particular, the required level of unfailing operation probability and γ percentage resource[3]. The latter indicator is significant for military wheeled vehicles[4]. In this respect, it is a well-known fact that the automobile engines of ZIL, Ural, GAZ series have operational difficulties after a long downtime.

The results of research carried out by our and other authors [2,3] prove that the engine crankcase oil contains particles of iron, aluminum, lead, tin, and other metals due to machine elements wear. The mentioned metal particles of these metals, when mixed with crankcase oil, increase the adhesion properties of the oil and, as a direct consequence, the engine crankshaft starting speeds are reduced and do not meet the engine operating required speed (due to increased internal resistance). This phenomenon is especially observed in military transport vehicles during operations [7].

It is obvious, that the solution of the given problem is mainly conditioned by the average value of characteristics of random X variable:

$$\overline{X} = \frac{x_1+x_2+\cdots+x_n}{n} = \frac{\sum_{i=1}^{n} X_i}{n} . \tag{1}$$

From the root-mean-square deviation:

$$\sigma = \sqrt{\frac{\sum_{i=1}^{n}(x_i-\bar{x})^2}{n-1}} , \tag{2}$$

as well as from the index of variation:

$$V_x = \frac{\sigma}{\bar{x}} . \tag{3}$$

Exploratory research shows that the mileage of automobiles and their parts, as random values, the variation coefficient is [5] a minor variation $V < 0.1$, medium variation $0.1 < V < 0.33$ and major variation $V > 0.33$.

Besides the above-mentioned values describing random quantities, there is also the possibility of another characteristic situation - P. It is accepted in statistical analysis [1] that a probability of a situation is within $0 \le P \le 1$. When $P = 1$, the situation is referred to as "reliable," and when $P = 0$, the situation is referred to as "less probable."

Thus, the probability of unfailing operation of an automobile and its parts is determined by the following expression:

$$R_{(x)} = \frac{n-m(x)}{n} = 1 - \frac{m(x)}{n} , \tag{4}$$

where m (x) is the number of automobile failures or malfunctions or number of auto parts during an X run (time interval).

We accept that the probability of malfunction (failure) is the inverse of the unfailing operation and is determined by the following expression:

$$F_{(x)} = 1 - R_{(x)} = \frac{m(x)}{n} . \tag{5}$$

[3] GOST 27.002-2015, GOST 27.002-89. Nadezhnost' v tekhnike (in Russian).

[4] Tomasz Smal, Maintenance in Availability of Weapon Systems under Combat Operation - Optimization Possibilities. Advances in Military Technology, 2013.
https://www.researchgate.net/publication/283837304_Maintenance_in_availability_of_weapon_systems_under_combat_operation_-_Optimization_possibilities

The probability of complex systems' and automobiles' unfailing operation is determined by the concept of γ percentage resource[5]. The γ percentage resource for automobiles and their units [5] is accepted as 0.85 (85%), 0.90 (90%), and 0.95 (95%). We can note that for automobile active safety systems the γ percentage resource is accepted as 0.95 (95%), for other systems it is from 0.85 (85%) to 0.90 (90%). This means that the value of the γ percentage resource for the automobile or its corresponding unit is greater than the X resource, or equal to the specified mileage value (time interval).

Let us consider the case, where the sequence of random variables has the law of normal distribution, that is, the probability of unfailing operation is formed under the influence of many factors (the automobile structure contains 15.0 thousand or more auto parts) [5], where the influence of each is very small. This assumption is also valid in the case of the law of large numbers.

In this case, the normal distribution function for maintenance or replacement of auto parts is as follows:

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} \cdot e^{-\frac{(x-\bar{x})^2}{2\sigma^2}} \quad . \tag{6}$$

That is, the probability of providing γ percentage resource will be:

$$R(x) = \frac{1}{\sigma\sqrt{2\pi}} \int_x^{\infty} e^{\frac{(x-\bar{x})^2}{2\sigma^2}} \cdot dx \quad . \tag{7}$$

This means that the frequency of maintenance and repair of automobiles and auto parts should be determined according to the expression (7), which means drawing up the list of the regulated works and labor-intensity based on the vehicle reliability limiting and from the point of view of reliability critical auto parts and maintenance list.

The automobile reliability map is formed according to the above-mentioned reliability indices, and on that basis, the regulated list of vehicle technical maintenance is formed.

Let us consider the failure characteristics of six auto parts, which are limiting the functional reliability of the working brake system of the GAZelle type minibus, as characteristics of random value distribution consistency (Table 1).

Table 1

	Detail name	Variation series min max (shift)	Average arithmetic value X (shift)	The mean square deviation of the variation series	Variation coefficient	Parameter of failure flow	Distribution curve law
1.	Anchor pin 3105-3501216	426 ÷629	517	160.27	0.31	$1.93.10^{-3}$	Normal
2.	Brake pad 3302-3501090	6 ÷32	21	18.27	0.87	$4.76.10^{-2}$	Normal logarithm
3.	Brake Cylinder piston 3105-3501186	304 ÷458	396	83.16	0.21	$2.52.10^{-3}$	Normal
4.	Brake cylinder seal 3105-3501194	257 ÷316	307	58.33	0.19	$3.25.10^{-3}$	Normal
5.	Corrugated pipe Cover 3105-3501188	205 ÷617	421	412.58	0.98	$2.37.10^{-3}$	Exponential
6.	Caliper pistons 3105-3501190 right 3105-3501191 left	457 ÷731	578	132.94	0.23	$1.73.10^{-3}$	Normal
7.	Brake disc 3302-3501078	324 ÷425	384	82.9	0.21	$2.82.10^{-3}$	Normal

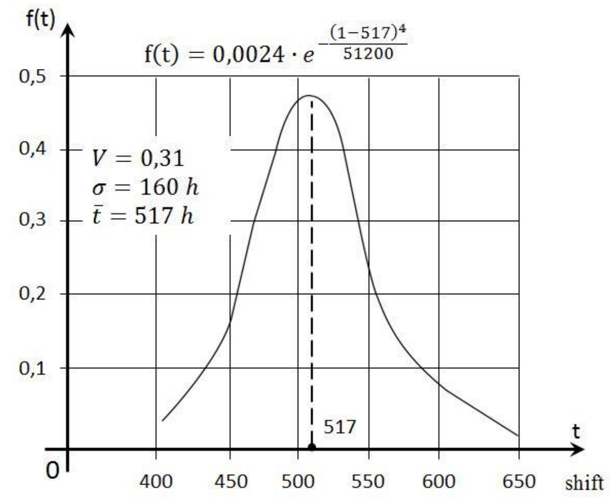
The distribution curves are shown in Fig. 1, otherwise known as the reliability map.

In Table 1 the so-called failure flow parameters ω (x) are presented according to the following expression:

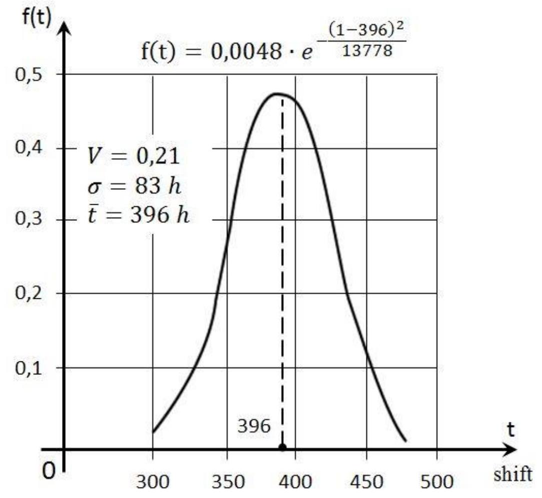
[5] GOST 27.002-2015, GOST 27.002-89. Nadezhnost' v tekhnike (in Russian).

$$\omega\ (x)\sum_{k=1}^{\infty} f_k\ (X) \tag{8}$$

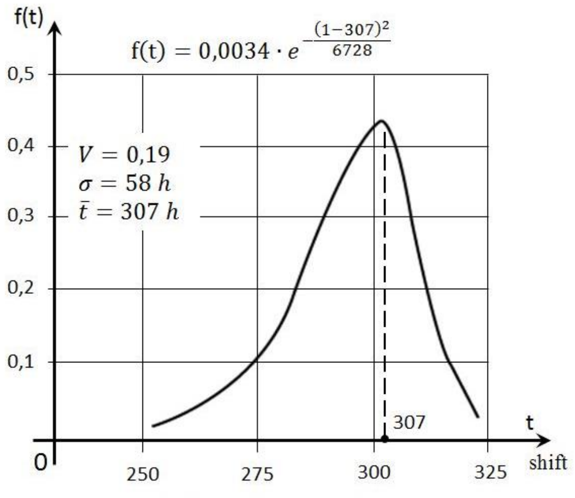
where f(x) is the density of failure formation probability, k is the ordinal number of failed auto parts. A more simplified failure rate parameter is the number of automobile operation failures per unit time, fail./km. Limiting the failure rate parameter on the explicit values, it is possible to adjust the resource value of the auto parts to a certain level of unfailing operation probability. In the above example, for the braking system, it is limited to 0.93, and accordingly, the resource of auto parts is determined. For the auto parts limiting the reliability of the brake system, it is shown in Fig. 2.



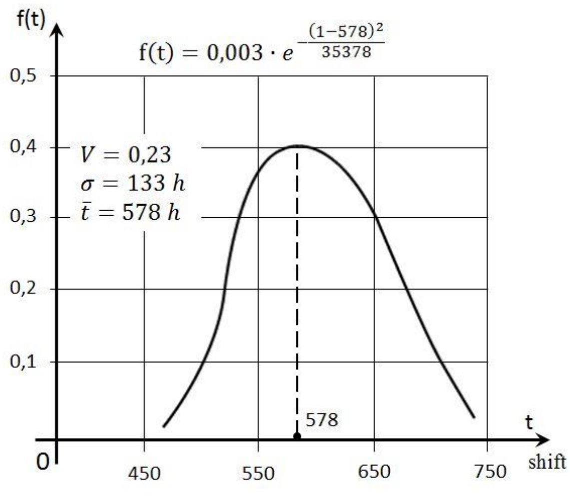
Anchor pin



Brake cylinder piston



Brake cylinder seal



Caliper pistons

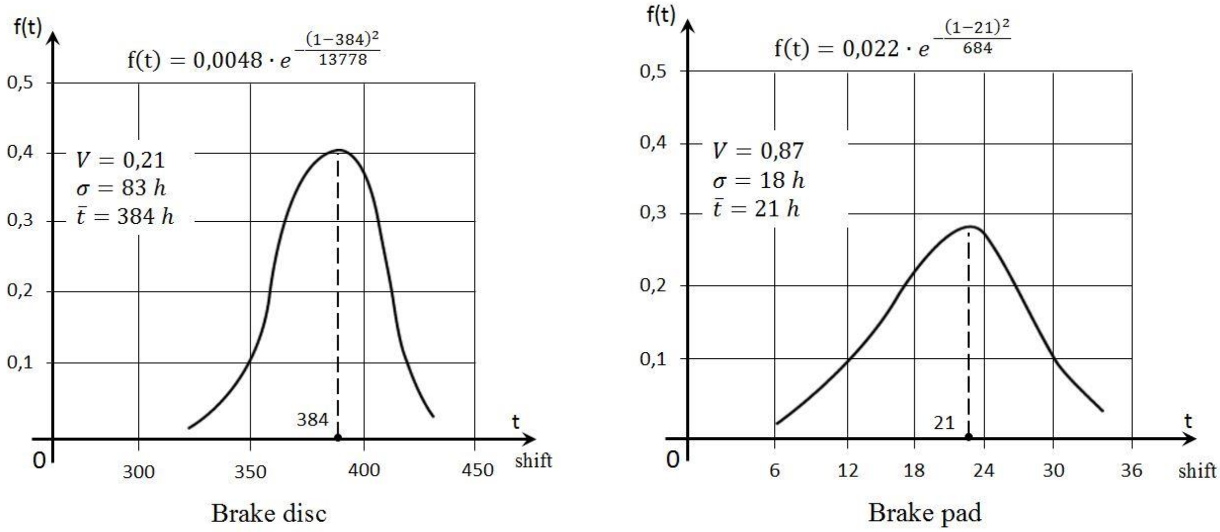
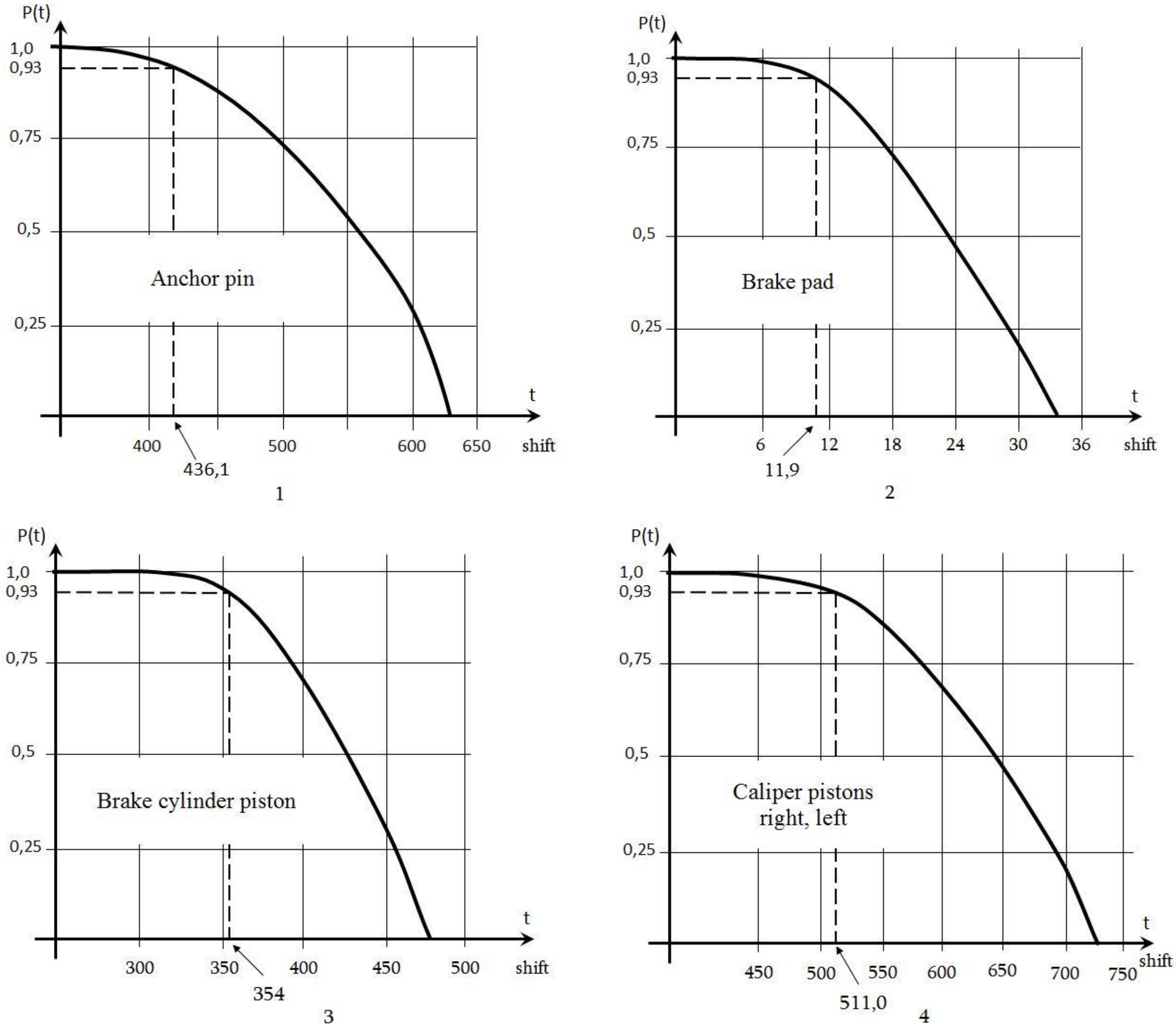


Fig. 1. *Functional failure distribution curves of GAZ-322133 minibus disc brake assembly*



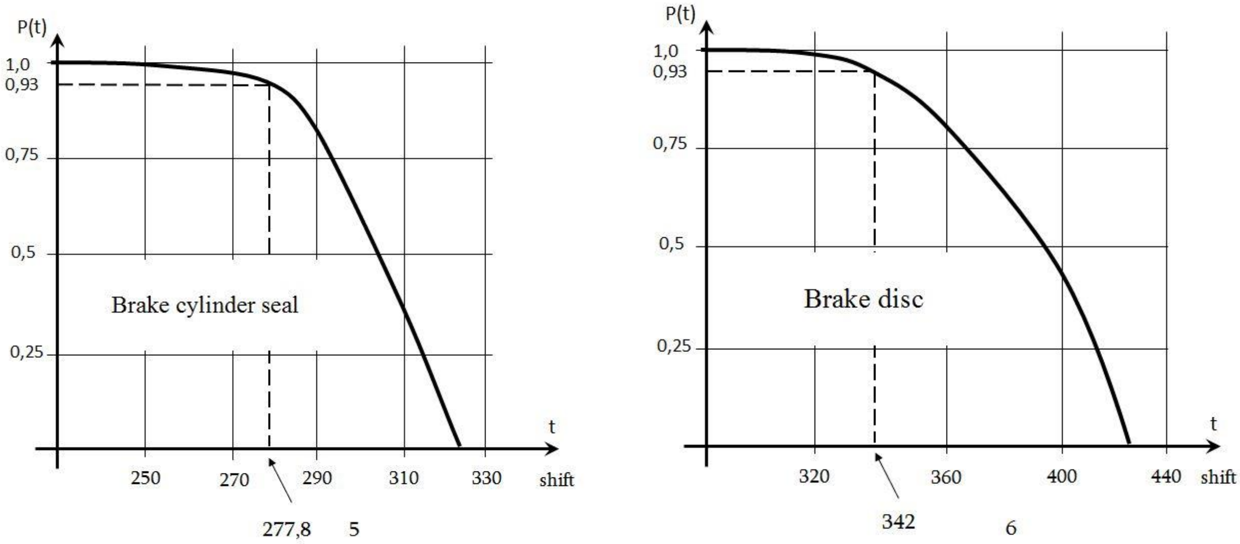


Fig. 2. *Auto part resource limiting the reliability of GAZ-322133 type minibus disc mechanism*

It follows from the above that the essence of an alternative random strategy for the exploitation of commercial and military vehicles is the amount of the resource for the actual regulated value of the probability of unfailing operation, after which it may be changed by a new one.

The advantage of regulated work based on a random strategy over a planned strategy is the traffic safety of vehicles. To achieve this goal, the regularities of failure and malfunction distribution were examined, identifying the features of reliability (mainly no-failure and longevity) using State Standard 27.002-2015 and State Standard 27.002-89 (Reliability in Technology)[6]. These standards operate in the Republic of Armenia according to the intergovernmental agreement (RF and RA), the relevant protocol, No. 84 P, 28.12.2015.

Consider the appropriateness of using a random strategy for commercial automobiles and military wheeled vehicles according to the dominant indicator. The dominant indicator for commercial automobiles is the probability of unfailing operation during the entire period of exploitation. It is formed by the automotive industry as per the scientific-technological progress. It persists through rational technical operation modes with the available technology and actual quality of the exploitation materials. Considering the current automotive equipment development level, the ordinary nature of the failures, the random strategy of technical condition restoration corresponds to the description of the stationary flow of failure recovery.

The γ percentage resource is the dominant indicator for wheeled military vehicles guaranteed for the given transport facilities. It is well known that during battle and other combat missions, the entire wheeled military vehicle must exhibit the highest coefficient of technical readiness. Military vehicles have not been used in a long time, but the list of regulatory works and a strict regular frequency ensure high technical readiness[7]. Studies show that the service of a certain number of auto parts is left out of the list of scheduled work, so it is necessary to study the most significant reliability indicator, which in this case is maintenance [8,9]. The performance of the studies is ensured according to State Standard[8] requirements, where the accuracy of the results is up to 0.95. The studies should be carried out within two years in the conditions of actual operation of the vehicles to ensure a high level of reliability of the statistical data and a high level of adequacy of the regularities obtained.

[6] GOST 27.002-2015, GOST 27.002-89. Nadezhnost' v tekhnike (in Russian).

[7] Voyenno-tekhnicheskoye informirovaniye. Plan-konspekt komandirskoy podgotovki po voyenno-tekhnicheskoy podgotovke. Organizatsiya ekspluatatsii, remonta i khraneniya avtomobil'noy tekhniki, 2019 (in Russian). https://shtab.su/konspekt/voenno-texnicheskoe-informirovanie/organizaciya-ekspluatacii-avtomobilnoj-texniki.html

[8] GOST 27.002-2015, GOST 27.002-89. Nadezhnost' v tekhnike (in Russian).

Conclusion

Based on the current situation, let us consider an alternative concept of the technical operation of commercial automobiles and wheeled military vehicles, which is based on a random strategy and implemented by the list of regulatory work according to the guaranteed value of the γ percentage resource.

- The random vehicle technical recovery strategy is based on the principle of stationary flow recovery of ordinary failures (mainly wear).
- It is known that the operating conditions of the vehicles, the quality of staff (drivers, repairmen, service specialists), the qualitative inhomogeneity of automobiles and auto parts, and the parameters characterizing the technical condition under the influence of other factors, the intensity of their change turns out to be very different.
- If the factor describing the technical condition of the automobile is assigned to Y_P, then for different automobiles the mileage (time interval) to reach such a condition will be l_{i1}, l_{i2} ...... lin, thus the no-failure millage (time) is obtained as a variation series. This raises the question of when and at what millage value to perform a technical condition control or maintenance operation.

As a result of the research, it is possible to develop and guarantee to invest:

1. the most efficient technical operation regulation modes of commercial vehicles developed according to a random strategy, which will ensure a reduction in materials and labor costs,
2. on the basis of modern automobile structural characteristics, significant changes and additions to the list of regulatory works for wheeled military vehicles based on the technical operation random strategy.

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Introduction should include a review of modern Armenian and foreign scientific achievements in the subject area, research and the results on which the work is based (Literature review). The literature review should emphasize the actuality and novelty of the issues considered in the study, on the basis of which the goals and objectives of the given work are set and described. The reference numbers to the source cited in the text are placed in square brackets strictly in sequence like [1], [2], ...

Introduction should contain information that will allow the reader to understand and evaluate the novelty and actuality of the research results presented in the article.

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Conclusion contains a brief description of the *Materials and Methods* section, as well as a brief statement of the research results. Here in compressed form, the main thoughts of the *Results and Discussion section* are repeated. In this section, it is necessary to compare the results obtained with the goal indicated at the beginning of the work. In Conclusion, the results of comprehension of the topic are summarized, conclusions, generalizations are made, and recommendations arising from the work are given, their practical significance is emphasized. In the final part of the article, it is desirable to include the prospects for the development of the research in this area.

Acknowledgments (if necessary)

This section, we refer to those persons who have assisted in the implementation of the study and those organizations that provide financial assistance.

References

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